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APPLICATIONS FOR ADMISSION AND TRANSFER

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	Discussion closes
Engineering Features of the Illinois Waterway. *Walter M. Smith*.....Oct., 1931	
Discussion. (Author's closure).....Feb., Mar., 1932, Mar., 1933	Closed
Determination of Principal Stresses in Buttresses and Gravity Dams. *W. H. Holmes*.....Jan., 1932	
Discussion. (Author's closure).....May, Aug., Oct., Dec., 1932, Mar., 1933	Closed
Stresses in Reinforced Concrete Due to Volume Changes. *C. P. Vetter*.... Feb., 1932	
Discussion .....May, Sept., 1932	Closed
The Compensated Arch Dam. *A. V. Karpov*.....Apr., 1932	
Discussion .....Oct., 1932, Mar., 1933	Closed
Development of Hydro-Electric Power as an Aid in Irrigation. *Charles C. Cragin* .....Apr., 1932	
Discussion .....Oct., 1932	Closed
Improvement of Hudson River by Narrowing the Navigable Fairway. *Lynne J. Bevan* .....May, 1932	
Discussion. (Author's closure).....Oct., 1932, Mar., 1933	Closed
Application of Duration Curves to Hydro-Electric Studies. *G. H. Hickox and G. O. Wessenauer*.....May, 1932	
Discussion. (Authors' closure).....Oct., Nov., 1932, Jan., Mar., 1933	Closed
George Washington Bridge: General Conception and Development of Design. *O. H. Ammann*.....Aug., 1932	
Discussion .....Nov., 1932, Jan., Feb., 1933	Mar., 193
Forests and Stream Flow. *W. G. Hoyt and H. C. Troxell*.....Aug., 1932	
Discussion .....Sept., Nov., Dec., 1932, Feb., Mar., 1933	Mar. 193
Tests of Riveted and Welded Steel Columns. *Willis A. Slater and M. O. Fuller* .....Sept., 1932	
Discussion .....Jan., Feb., 1933	Mar., 193
The Martinez-Benicia Bridge. *W. H. Kirkbride*.....Sept., 1932	
Discussion .....Nov., 1932, Feb., 1933	Mar., 193
Pre-Qualification of Contractors. *C. J. Tilden*.....Sept., 1932	
Discussion .....Nov., 1932, Jan., 1933	Mar., 193
Tests for Hydraulic-Fill Dams. *Harry H. Hatch*.....Oct. 1932	
Discussion .....Jan., Feb., Mar., 1933	Apr., 193
George Washington Bridge: Organization, Construction Procedure, and Contract Provisions. *Edward W. Stearns*.....Oct., 1932	
Discussion .....Dec., 1932	Apr., 193
Foundation Treatment at Rodriguez Dam. *Charles P. Williams*.....Oct., 1932	
Discussion .....Mar., 1933	Apr., 193
A History of the Development of Wooden Bridges. *Robert Fletcher and J. P. Snow* .....Nov., 1932	
Discussion .....Jan., Feb., Mar., 1933	Apr., 193
Distribution of Shear in Welded Connections. *Henry W. Troelsch*.....Nov. 1932	
Discussion .....Feb., Mar., 1933	Apr., 193
Work of Rivets in Riveted Joints. *A. Hrennikoff*.....Nov., 1932	
Discussion .....Mar., 1933	Apr., 193
Study of Stilling-Basin Design. *C. Maxwell Stanley*.....Nov., 1932	Apr., 193
A Problem of Soil in Transportation in the Colorado River. *S. L. Rothery*..Dec., 1932	May, 193
George Washington Bridge: Design of Superstructure. *Allston Dana, Aksel Andersen, and George M. Rapp*.....Dec., 1932	May, 193
George Washington Bridge: Design of the Towers. *Leon S. Moisseiff*.....Dec., 1932	May, 193
Model Law for Motion of Salt Water Through Fresh. *Morrough P. O'Brien and John Cherno*.....Dec., 1932	
Discussion .....Mar., 1933	May, 193
Wind Stress Analysis Simplified. *L. E. Grinter*.....Jan., 1933	May, 193
George Washington Bridge: Construction of Substructure. *Montgomery B. Case* .....Jan., 1933	May, 193
George Washington Bridge: Construction of Steel Superstructure. *E. W. Bowden and H. R. Seely*.....Jan., 1933	May, 193
Progress Report of the Special Committee on Meteorological Data.....Jan., 1933	May, 193
Evaporation from Water Surfaces: A Symposium.....Feb., 1933	May, 193
George Washington Bridge: Materials and Fabrication of Steel Structure. *Herbert J. Baker*.....Feb., 1933	May, 193
George Washington Bridge: Approaches and Highway Construction. *T. C. Evans*.....Feb., 1933	May, 193

CONTENTS FOR MARCH, 1933

PAPERS

PAGE

Developments in Reinforced Brick Masonry.
By James H. Hansen, Jun. Am. Soc. C. E. 407

The Plan of Boston, Massachusetts: A Capital City.
By Arthur C. Comey, M. Am. Soc. C. E. 429

DISCUSSIONS

Engineering Features of the Illinois Waterway.
By Walter M. Smith, M. Am. Soc. C. E. 441

Determination of Principal Stresses in Buttresses and Gravity Dams.
By W. H. Holmes, Assoc. M. Am. Soc. C. E. 449

The Compensated Arch Dam.
By Eugene Kalman, M. Am. Soc. C. E. 457

Improvement of Hudson River by Narrowing the Navigable Fairway.
By Lynne J. Bevan, M. Am. Soc. C. E. 477

Application of Duration Curves to Hydro-Electric Studies.
By G. H. Hickox and G. O. Wessenauer, Juniors, Am. Soc. C. E. 480

Forests and Stream Flow.
By Messrs. W. C. Lowdermilk, Rhodes E. Rule, and Robert E. Kennedy 484

Tests for Hydraulic-Fill Dams.
By Messrs. Jeptha A. Wade, and Stanley M. Dore 495

Foundation Treatment at Rodriguez Dam.
By Messrs. L. A. Robb, Francisco Gomez-Perez, and F. A. Noetzli 503

A History of the Development of Wooden Bridges.
By Messrs. C. J. Hogue, William A. Oliver, Philip G. Laurson, Richard S. Kirby, Wells N. Thompson, Benjamin Wilder Guppy, Ivan E. Houk, and William H. Baker 510

CONTENTS FOR MARCH, 1933 (*Continued*)

PAGE

Distribution of Shear in Welded Connections.
By W. H. Jameson, Assoc. M. Am. Soc. C. E. 521

Work of Rivets in Riveted Joints.
By Messrs. Henry W. Troelsch, Henry B. Seaman, A. H. Finlay, and F. P. Shearwood 523

Model Law for Motion of Salt Water Through Fresh.
By W. E. Howland, Assoc. M. Am. Soc. C. E. 531

For Index to all Papers, the discussion of which is current in PROCEEDINGS, *see page 2*

The Society is not responsible for any statement made or opinion expressed in its publications

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SOCIETY AFFAIRS

Annual Report of the Board of Direction for the Year Ending December 31, 1932 . . . . . following page 534

MEMBERSHIP

Applications for Admission and Transfer .....following page 24 of Society Affairs

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

DEVELOPMENTS IN REINFORCED BRICK MASONRY

By James H. Hansen[1], Jun. Am. Soc. C. E.

Synopsis

This paper contains a brief history of the development of reinforced brick masonry to the time that engineers in the United States became interested in the subject. Its development in this country is then given in detail by analysis of tests.

Reinforced brickwork is a system of construction wherein reinforcing bars are placed in the joints of brick masonry (as shown in Table 1, Column (2)) so that they will take tensile stresses. Otherwise, brick masonry is built in the customary manner.

It is not the purpose of this paper to offer fully developed theories and conclusions, but to set down such information as is available to stimulate interest and discussion.

Historical Notes

To that brilliant engineer, Marc Isambard Brunel, is generally conceded the credit for the invention of reinforced brick masonry. He was knighted by Queen Victoria in recognition of his greatest engineering feat, the Thames Tunnel.

The first step in connection with the building of the tunnel was the construction of a shaft, which Mr. Richard Beamish (who was Resident Engineer on the work), describes[2] as being 50 ft in diameter and 42 ft in height. It had forty-eight wrought-iron bolts 1 in. in diameter built into the brickwork, which were attached to wooden curbs at the bottom and top with nuts and screws, "thus binding the work into a solid mass". Iron hoops and timber bonds, 9 in. deep and $\frac{1}{2}$ in. thick, were also let into the brickwork at intervals as work on the shaft progressed. When it had attained the height of 42 ft,

Note.—Discussion on this paper will be closed in August, 1933, *Proceedings*.

[1] Warren, Cornwall Bridge P. O., Conn.

[2] "Memoir of the Life of Sir Marc Isambard Brunel", p. 211; see, also *Civil Engineering*, September, 1931, Vol. 1, No. 12, p. 443.

the shaft was allowed to sink into the earth by excavating the gravel from the interior. Its strength and cohesion were severely tested when it suddenly descended 7 in. on one side where it had entered some soft ground, while on the other side, where the ground remained hard, it sank only 3½ in. "The surge was alarming", says the annotator, "but so admirably was the structure bound together that no injury was sustained". After being sunk, the shaft was completed to the required depth by underpinning, but the shaft for the opposite shore was constructed in the same manner as the first, for its entire height of 70 ft. The second shaft differed from the first one in being slightly conical to facilitate its sinking, and it was more liberally supplied with reinforcing material.

According to Mr. Beamish the fortunate combination of iron ties with bricks and cement in the construction of the shaft of the Thames Tunnel suggested to Mr. Brunel the idea of establishing a general principle of construction "which should unite in an eminent degree utility with economy". Accordingly, after a series of experiments, he erected a semi-arch in the tunnel yard, without the use of centering, to a half-span length of 60 ft, and with a rise of only 10 ft 6 in. He counterbalanced it with weights totaling 62 700 lb. In the first part of this experiment, ties of fir, and, subsequently, hoop iron, were used. The arch stood for two years when it collapsed because some one dug too close to its shallow foundations. About the same time, Sir Isambard built what was later known as the "Nine Elms Beam" which was 57 in. deep, with an inverted **T**-section, 24 in. wide for the lower 18 in. and 19 in. wide for the upper part. This beam had a clear span of 21 ft and contained seventeen 1¼-in. by ⅛-in. iron bands, on which was placed a load of 24 000 lb. It stood for two years when it was tested to destruction, in 1838, at a load of 68 326 lb, including the dead load. Failure was caused by the weakening of the iron straps, most of which were broken.

By this time other engineers were experimenting with this type of construction. In 1837, Col. C. W. Pasley, of the Royal Engineers, conducted a series of tests on brick beams, and in 1851, at the London Building Exhibition, a beam somewhat similar to the "Nine Elms Beam" was tested as one of the first demonstrations of the strength of Portland cement. This beam carried about five times as great a load as the "Nine Elms Beam", the latter having been built with Roman cement. It is interesting to note that these early tests were conducted to determine the quality of the various cements and mortars; the experimenters evidently did not question the adaptability of brick for this type of construction. Mention of its use is made in English publications in connection with several large buildings constructed at that time, and Isambard Kingdom Brunel, son of Sir Isambard Brunel, built one of his most beautiful brick arch bridges with hoop iron set in the brickwork in the manner which his father's experiments had proved to be successful.

From this stage it was only a step to the development of reinforced concrete, and this step taken, reinforced brickwork, which had played such a large part in introducing reinforced concrete, was allowed to languish and was practically forgotten. Mention was made of it now and then in English

technical publications[3] during the last hundred years, but it was not until 1922 that it emerged from obscurity. In that year, A. Brebner, Under Secretary, Government of India, Public Works Department, published two volumes entitled "Notes on Reinforced Brickwork"[4], from which it is learned that this type of construction had had a rather wide use in India, where its development was due in part to the high cost of form lumber and concrete material. There was a lack of workmen familiar with concrete construction, whereas there was an abundance of brick and brick-layers. Several of the local governments in India have adopted it in the construction of public buildings and it has been utilized in the construction of many private buildings. In all, nearly 3 000 000 sq ft of reinforced brickwork was laid in the three years prior to 1922. Mr. Brebner's volumes include reports of tests conducted on numerous types of slabs, beams, cantilevers, etc., and, as a basis of comparison, similar tests on reinforced concrete structures, from the results of which he concluded that the theory of reinforced concrete design would apply to reinforced brick masonry.

In 1924, Capt. T. W. D. Miller, of the Royal Engineers, published a paper[5] written from notes made on the work done in the erection of an officers' hostel at Chaklala, India, in which he describes in detail the design and construction of the roof slabs of reinforced brick masonry. The slabs were originally designed for tile and reinforced concrete, but it was considered that the cost would be reduced by substituting reinforced brick masonry. The design is based on the standard reinforced concrete formulas with compressive stresses in the masonry limited to 600 lb per sq in. and tensile stresses in steel to 14 000 lb per sq in.

Dr. Engr. Shigeyuke Kanamori, Civil Engineer of the Department of Home Affairs of the Imperial Japanese Government, writing on the subject of reinforced brick masonry[6] states that he has built many structures, such as buildings, retaining walls, quay walls, bridge piers, abutments, etc., of reinforced brick masonry. His design practice is to use reinforced concrete formulas with the value of 25 for n, the ratio of elasticity moduli, and brick made with a slot in one side which always coincides with the slot below, providing space for the rods.

Characteristics of Brick and Brick Masonry

The books and articles cited are recommended for perusal, but are not taken into consideration in this paper, as its object is the discussion of the technical phases of reinforced brick masonry as developed in the United States. It has been considered best in this country to make independent experiments so as to avoid the difficulties incident to allowances for differences in climatic conditions and materials, the principal difference being that the brick in the United States are smaller and stronger than those made in India and Japan. The tests and experiments discussed in this paper were by no

[3] *Engineering* (London), July, 1872.
[4] *Technical Paper No. 38*, Govt. of India, Public Works Dept.
[5] *Royal Engineers Journal*, Vol. 38, p. 108.
[6] *Brick and Clay Record*, July 15, 1930.

means ideally planned, but they include a variety of beams and slabs over a number of span lengths, and, for practical purposes, they establish the truth of certain necessary assumptions and values that can be used with safety subject to further development. The necessary determinations were: (*a*) The compressive, tensile, and shear strengths of brick, mortar, and brick masonry; (*b*) the modulus of elasticity of brick masonry; (*c*) its resistance to shear; and (*d*) the bond strength developed by bars in mortar joints.

Compressive Strength.—The Building Code Committee of the United States Department of Commerce has distributed tables[7] of brick masonry strengths upon which their allowable stresses were based. The maximum compressive strength listed was 18 337 lb per sq in., the minimum, 1 659 lb per sq in., and the average of all is 6 940 lb per sq in. Only 27% had compressive strengths of 3 500 lb per sq in., or less, and these average 3 115 lb per sq in.[8] The National Bureau of Standards has tabulated other properties of brick[9], which show that: (1) The direct ratio of compressive strength, flat, of a half brick to compressive strength on edge is 1,08; and (2) the inverse ratio of compressive strength, flat, to the modulus of rupture is 0.24; to tensile strength, 0.09; and to shear strength, 0.39. These are averages of fifty tests of brick from four different parts of the United States. However, there is a considerable variation of these ratios for brick in different territories.

Handbooks and test reports give widely different values for the compressive strength of brick masonry, which, considering the number of variables, is only to be expected; but it can be controlled and predetermined within limits just as it is in other construction materials where these factors are known. The size of the specimen tested, the type of workmanship, the mortar, and the brick strength are of prime importance for any direct comparison. Investigators of brick masonry have generally confined themselves to tests on individual bricks, "wallettes", approximately 18 by 34 in. high, and wall panels, 6 by 9 ft high, with a few tests on small piers of various sizes, some as small as 8 by 12 in. Which of these specimens gives results most applicable to the strength of masonry in beam action has not been determined, but perhaps a brief summary of them all will suggest an answer.

In general, if all other things are equal, the relation between strength of the masonry and strength of brick is a straight-line ratio until the ultimate strength of mortar is reached. The Bureau of Standards reports[10] the average strength of "wallettes", 18 by 34 in. high, with 1:0.1:3 cement mortar, to be 2 110 lb per sq in. It reports[11] the strength of walls to be 86% of the strength of "wallettes", and it is generally conceded that walls 9 ft high will develop a compressive strength equal to from 25 to 30% of the brick strength.

[7] "Compression Tests on Brick Masonry": A Compilation of Data Covering 708 Individual Tests of Brick Masonry in the Form of Piers and Walls", March, 1926.

[8] Discussion by Maj. L. B. Lent of the proposed recommendations of the Building Code Committee, U. S. Department of Commerce, May 25, 1928. (Not published.)

[9] "Compressive Strength of Clay Brick Walls", by A. H. Stang, D. E. Parsons, and J. W. McBurney, *Research Paper No. 108*, National Bureau of Standards.

[10] *Research Paper No. 108*, National Bureau of Standards, U. S. Dept. of Commerce, Table 23.

[11] *Loc. cit.*, Fig. 35.

When small piers $\left(\frac{\text{height}}{\text{diameter}} = 2\right)$ are tested, the strength of brick masonry laid in cement mortar will no doubt be greater. After a study of hundreds of tests of the nature of the foregoing, the Building Code Committee of the Department of Commerce has recommended[12] that the allowable working stress in direct compression on the best brick masonry, when properly designed, should be increased to 600 lb per sq in. Allowable working stresses at present seldom exceed 350 lb per sq in.

Modulus of Elasticity.—The modulus of elasticity of brick masonry has been variously determined as follows:

Authority	Modulus of Elasticity, in Pounds per Square Inch
T. G. MacCarthy[13]	3 000 000
A. H. Beyer, M. Am. Soc. C. E., and W. J. Krefeld[14] Assoc. M. Am. Soc. C. E.	1 628 830
Mason Vaugh[15]	3 171 000
M. J. Feeney and R. W. Miller[16]	1 350 000
J. G. Bragg[17]	580 000 to 3 500 000

The values cover ranges of mortar mixes generally used.

Messrs. A. H. Stang, D. E. Parsons, and J. W. McBurney have stated[18] that there is no value for the modulus of elasticity that is constant over a large stress range, but they give calculated values for the secant modulus over a stress range of 0 to 250 lb per sq in. for cement mortar walls to be from 727 000 lb per sq in. to 2 652 000 lb per sq in. These variations are not alarming and can be controlled more or less as with concrete, which has a variation of 750 000 lb to 3 000 000 lb, resulting in values of n of from 10 to 40.[19]

Tensile Strength.—Little has been published concerning the tensile strength of brick masonry; it is limited in the case of cement mortar by the bond strength between brick and mortar and in the case of cement-lime mortar by the tensile strength of the mortar itself. Messrs. L. A. Palmer and J. V. Hall give[20] bond strengths of 31.7 lb for 1:1:6 cement-lime mortar and approximately the same value for 1:3 cement mortar. Another authority[21] gives 10 to 40 lb (including a factor of safety of from 8 to 10) as the tensile strength of brick masonry. Many authorities agree that the more porous

[12] "Modifications in Recommended Minimum Requirements for Masonry Wall Construction", March 16, 1931.

[13] Rept. on Brick Masonry, Southern Methodist Univ. (Unpublished).

[14] *Engineering and Scientific Papers No. 12*, Columbia Univ., April, 1923, p. 58.

[15] *Bulletin No. 37*, Univ. of Missouri, Vol. 29.

[16] In a thesis presented to Mass. Inst. Tech. in partial fulfillment of the requirements for the degree of Bachelor of Science in Architectural Engineering.

[17] *Technologic Paper No. 111*, National Bureau of Standards, U. S. Dept. of Commerce.

[18] *Research Paper No. 108*, National Bureau of Standards, Washington, D. C.

[19] Concrete Engineer's Handbook, by Hool and Johnson, Vol. I, p. 6, First Edition, 1920.

[20] *Research Paper No. 290*, National Bureau of Standards, Washington, D. C.

[21] Kidder's Architects and Builders Handbook, Seventeenth Edition, 1921, p. 179.

bricks develop higher bond strengths, provided they are properly wetted before laying. Ordinary bricks that are very dry will absorb the water from the mortar too rapidly, and those completely saturated lack the suction necessary to cause the mortar to cling to them. This is also slightly noticeable with dense, hard bricks of low absorption.

Comparison of Test Data

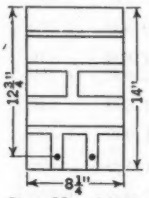
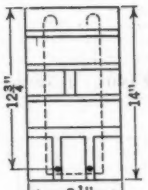
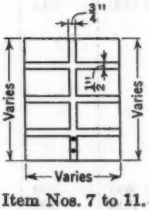
Referring to Table 1, complete information regarding the conditions of each test was not always available and certain assumptions have been made; but, in general, all data that would materially affect the results obtained were taken from the test reports. The calculations are based on the assumption that the value of 20 for n, the ratio of elasticity moduli, would give a fairly accurate distribution of stresses, and that the theories of beam action in composite construction, as defined in the report of the Joint Committee on Standard Specifications for Reinforced Concrete[22], would apply. Due to the methods used in some of the beam tests, the testing apparatus itself placed a load of from 240 lb to 500 lb on the beam. However, as the true weight of this item was not reported in all cases, it was not considered; only live loads are listed. The weight of the beam itself was also disregarded in calculating the bending moment, and stresses in the steel and brickwork. The inclusion of these items would raise the stresses, but not enough to affect the conclusions as to the stability of the beams. In calculating the shear, unit shear, and bond (Columns (6), (9), and (10), Table 1), however, the weight of the beam was calculated at 125 lb per cu ft and used because these stresses are critical and it is important to know the maximum value that can be developed with safety. In Table 1 the calculated stresses are placed opposite the loads to which they apply, which are: (1) Load at first crack or at a deflection of $\frac{1}{360}$; (2) load at which the last deflection was measured; and (3) the ultimate load.

Items Nos. 1 to 6, Table 1.—The summary of test results by Mr. Vaugh pertains to a group of beams with clear spans of 12.00 ft. The distance between the two axle loads was 4.00 ft in each case. Three of the beams were made up of a 1:0.0:2 cement-sand mortar (Items Nos. 1, 2, and 3) and were tested at 49, 44, and 42 days, respectively. The remaining three (Items Nos. 4, 5, and 6), were made up of a cement-lime-sand mortar in the ratio of 1:0.1:3 and were tested at 41, 40, and 46 days, respectively. In Items Nos. 4 and 5, $\frac{1}{4}$-in. round bars—and in Item No. 6, No. 9 wire—spaced 8.5 in., on centers, were used as stirrups. Two $\frac{3}{4}$-in. round, plain bars were used for reinforcement as shown in Column (2). All rods were hooked except those in Item No. 1.

Mr. Vaugh states[15] that only one beam (Item No. 3) failed by shear (at 69 lb per sq in.). The other two with no shear reinforcement failed in bond, or tension in the longitudinal steel, despite the fact that shear stresses were as much as 85 lb per sq in. (see Item No. 1). However, when stirrups and cement-lime mortar were used in the same type of beam (Items Nos. 4, 5, and 6), the failure was due to tension in all cases. In two cases (Items Nos.

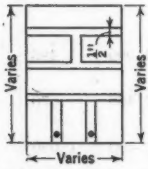
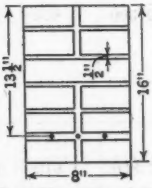
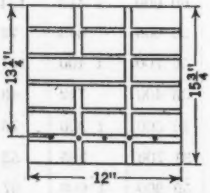
[22] *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1153.

TABLE 1.—CHARACTERISTICS OF REINFORCED BRICK BEAMS AND RESULTS OF TESTS (ALL BEAMS FREELY SUPPORTED AT ENDS)

Item No.	Cross-sections	Live load, in thousands of pounds (N = total uniform load; remainder are axle loads)	Deflection, in thousandths of an inch	Bending moment, in thousands of inch-pounds	Total maximum shear, V in pounds	Stresses, in Pounds per Square Inch: Tensile stress, f_s, in reinforcement	Compression stress, f_b, in beams	Shear, v	Bond strength, u
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	MASON VAUGH								
	 Item Nos. 1 to 3.								
1		7.0	93	....	.....	.....	.....	....	.....
		14.0	309	336	7 672	34 900	1 495	85	150
2		4.0*	30	....	.....	.....	.....	....	.....
		7.0	84	168	4 172	17 400	670	46	82
3		7.0*	84	....	.....	.....	.....	....	.....
		11.0	182	264	6 172	27 400	1 050	69	112
	 Item Nos. 4 to 6.								
4		7.0*	91	....	.....	.....	.....	....	.....
		17.0†	364	407	9 172	42 300	1 620	102	180
5		7.0‡	194	....	.....	.....	.....	....	.....
		11.2‡	533	268	6 247	27 800	1 060	70	122
6		7.0‡	89	....	.....	.....	.....	....	.....
		17.0‡	387	407	9 172	42 300	1 620	102	180
	M. J. FEENEY AND R. W. MILLER								
7	 Item Nos. 7 to 11.	9.5†	60	....	.....	.....	.....	....	.....
		11.5	96	....	.....	.....	.....	....	.....
		18.5	....	185	9 340	57 800	1 610	136	369
8		9.5†	128.5	90	4 675	33 100	820	84	214
		12.5	187.5	....	.....	.....	.....	....	.....
		16.4	....	163.5	8 350	60 000	1 485	154	384
9		7.5†	33	75	3 925	26 000	643	72	170
		12.5	95	....	.....	.....	.....	....	.....
		16.3	....	163	8 325	56 700	1 400	151	360
10		5.5†	126.5	88	3 030	25 400	712	43	109
		7.5	227	....	.....	.....	.....	....	.....
	Item No. 7: 8¼ by 10½; d = 9.25.	10.1	....	162	5 330	46 500	1 320	76	192
	Item No. 8: 8 by 9; d = 8.25.								
	Item No. 9: 7½ by 10; d = 8.75.	5.5†	109.5	88	3 030	31 700	985	53	136
11	Item No. 10: 8 by 10½; d = 10.0.	8.5	263.5	....	.....	.....	.....	....	.....
	Item No. 11: 8 by 10; d = 8.12.	10.4	....	166	5 480	59 800	1 860	97	246

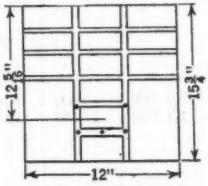
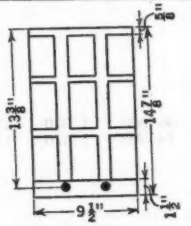
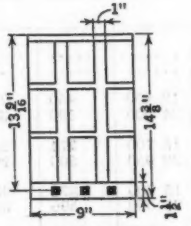
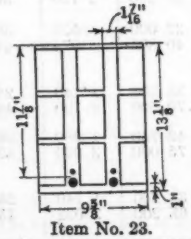
* No sign of failure. † Load at first indication of failure. ‡ Ultimate load.

TABLE 1.—(*Continued*)

Item No.	Cross-sections	Live load, in thousands of pounds (N=total uniform load; remainder are axle loads)	Deflection, in thousandths of an inch	Bending moment, in thousands of inch-pounds	Total maximum shear, V, in pounds	Stresses, in Pounds per Square Inch			
						Tensile stress, f_s, in reinforcement	Compression stress, f_b, in beams	Shear, v	Bond strength, u
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	M. J. Feeney and R. W. Miller (*Continued*)								
12		9.5†	28	95	4 950	25 700	680	66	167
		12.5	52	....	.....	.....	.....	....	....
		22.0‡	....	220	11 200	59 500	1 570	149	377
13		9.5†	....	95	4 950	24 800	646	64	162
		17.5‡	....	175	8 950	45 800	1 190	115	292
14		9.5†	42.5	95	4 950	24 800	646	64	162
		12.5‡	61	....	.....	.....	.....	....	.....
		19.5‡	....	195	9 950	51 000	1 330	128	324
15		5.0†	54	80	2 740	21 400	566	36	69
		10.5	174.5	....	.....	.....	.....	....	.....
	Item Nos. 12 to 16.	13.3‡	....	212	6 890	57 000	1 530	91	230
16	Item No. 12: 8 by 11½; d = 10.67.	1.5	3	....	.....	.....	.....	....	.....
	Item No. 13: 8 by 11½; d = 11.0.	1.5	5	....	.....	.....	.....	....	.....
	Item No. 14: 8 by 11½; d = 11.0.	1.5	7	....	.....	.....	.....	....	.....
	Item No. 15: 8 by 11½; d = 10.75.	3.8	45	....	.....	.....	.....	....	.....
	Item No. 16: 8 by 11½; d = 10.75.	3.8	32	....	.....	.....	.....	....	.....
		13.3	....	212	6 890	57 000	1 530	91	230
	Judson Vodges								
17		5.4†	....	87	3 250	47 300	593	33	110
		7.6‡	....	121	4 314	65 900	823	43	146
18		4.9†	....	79	3 010	43 000	537	30	102
		7.5‡	....	120	4 272	65 000	813	43	145
	Item Nos. 17 and 18.								
19		8.1†	....	129	4 837	43 300	599	33	101
		10.7‡	....	171.2	6 160	57 600	800	42	130
	Item No. 19.								

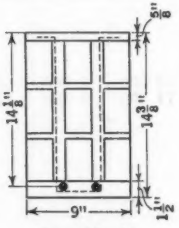
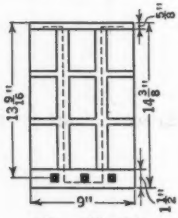
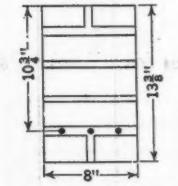
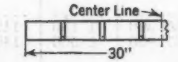
† Load at first indication of failure. ‡ Ultimate load.

TABLE 1.—*(Continued)*

Item No.	Cross-sections	Live load, in thousands of pounds (N = total uniform load; remainder are axle loads)	Deflection, in thousandths of an inch	Bending moment, in thousands of inch-pounds	Total maximum shear, V, in pounds	Stresses, in Pounds per Square Inch: Tensile stress, f_s, in reinforcement	Compression stress, f_c, in beams	Shear, v	Bond strength, u
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	Judson Vodges (*Continued*)								
20	 Item No. 20.	7.8† 13.0‡	....	125.2 208	4 712 7 300	44 800 74 300	666 1 108	35 54	105 162
	T. G. MacCarthy								
21	 Item No. 21.	7.0 15.0 16.2	59 140	 259	.... 8 588	.... 23 800	 814	 78	 148
22	 Item No. 22.	7.0 16.2	54 160	.... 259.5	..... 8 580	..... 24 000	 905	 81	 98
23	 Item No. 23.	7.0 19.0 24.0‡	87 270 380	.... 304 384	..... 9 932 12 432	..... 30 300 37 950	..... 1 180 1 500	.... 112 140	..... 137 173

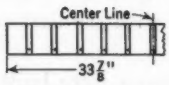
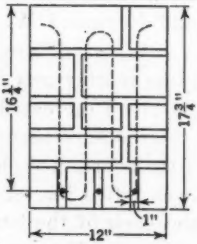
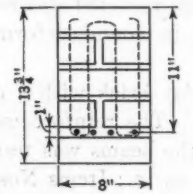
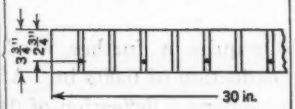
†Load at first indication of failure. ‡Ultimate load.

TABLE 1.—*(Continued)*

Item No.	Cross-sections	Live load, in thousands of pounds (N = total uniform load; remainder are axle loads)	Deflection, in thousandths of an inch	Bending moment, in thousands of inch-pounds	Total maximum shear, V, in pounds	Stresses, in Pounds per Square Inch: Tensile stress, f_s, in reinforcement	Compression stress, f_c, in beams	Shear, v	Bond strength, u
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	T. G. MacCarthy *(Continued)*								
24	 Item No. 24.	23.0†	250	368	11 988	33 700	1 169	109	208
		26.5‡	339	425	13 738	39 000	1 350	125	236
25	 Item No. 25.	7.0	62	....	.....	.....	.....	....	.....
		25.0	258	400	12 980	36 900	1 390	123	148
		30.0‡	340	480	15 480	44 300	1 670	147	176
	Hugo Filippi								
26	 Item Nos. 26 to 31.	2.0†	125	....	.....	.....	.....	....	.....
		2.5‡	344	....	.....	.....	.....	....	.....
27		3.0†	156	60.0	2 035	18 850	450	27	61
		4.0‡	187	79.2	2 524	24 900	590	33	75
28		2.5†	94	50	1 735	15 700	374	23	51
		3.3‡	125	66.6	2 201	20 950	500	29	65
29		2.5†	94	50.0	1 735	15 700	374	23	51
		4.0‡	250	80.0	2 535	25 000	596	34	75
30		2.5†	94	50.0	1 735	15 700	374	23	51
		7.5‡	219	150.0	4 285	47 300	1 120	56	126
31		3.5†	94	70.0	2 285	22 000	525	30	67
		6.5‡	391	130.0	3 785	40 800	975	49	111
32	 Item Nos. 32 to 37.	2.0 N†	156	17.3	1 150	38 200	1 140	25	150
		3.7 N‡	500	31.8	2 012	70 500	2 120	44	280
33		2.3 N†	156	19.3	1 285	42 750	1 280	28	178
		4.2 N‡	563	33.9	2 260	75 000	2 230	49	314
34	Width = 30 in; depth of slab = 2½ in.; effective depth, d = 1½ in.	2.0 N†	125	17.3	1 150	38 200	1 140	25	150
		3.8 N‡	594	30.8	2 048	68 200	2 032	45	285

† Load at first sign of failure. ‡ Ultimate load.

TABLE 1.—*(Continued)*

Item No.	Cross-sections	Live load, in thousands of pounds (N=total uniform load; remainder are axle loads)	Deflection, in thousandths of an inch	Bending moment, in thousands of inch-pounds	Total maximum shear, V, in pounds	Stresses, in Pounds per Square Inch			
						Tensile stress, f_s, in reinforcement	Compression stress, f_b, in beams	Shear, v	Bond strength, u
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	Hugo Filippi (*Continued*)								
35	 Item Nos. 38 to 43.	2.0 N†	125	17.3	1 150	38 200	1 140	25	150
		3.4 N‡	625	27.7	1 845	61 400	1 830	40	256
36		2.2 N†	313	19.1	1 270	42 200	1 259	28	177
		3.5 N‡	563	28.5	1 950	63 200	1 880	42	270
37		2.5 N†	219	21.0	1 400	46 500	1 390	30	192
		3.8 N‡	531	30.4	2 025	67 400	2 000	44	282
38		4.0 N†	344	58.7	2 440	33 800	955	26	123
		6.3 N‡	938	85.5	3 567	45 800	1 207	38	182
39		4.0 N†	344	58.7	2 440	33 800	955	26	123
		6.2 N‡	688	85.0	3 543	48 000	1 355	38	180
40		4.0 N†	313	58.7	2 440	33 800	955	26	123
		7.5 N‡	1 000	107.0	4 190	57 800	1 630	45	212
41		3.5 N†	219	52.6	2 190	30 300	855	23	111
		8.5 N‡	1 125	112.8	4 690	64 900	1 830	50	238
42		2.5 N†	78	40.6	1 690	23 300	659	18	86
		8.4 N‡	875	111.0	4 626	59 400	1 560	48	350
43		3.0 N†	125	46.6	1 940	26 800	755	21	99
		9.0 N‡	906	118.4	4 940	68 100	1 920	53	250
	Tests by the Writer								
44		1.0	0	....	.....	.....	.....	....	....
		3.1	32	....	.....	.....	.....	....	....
		5.3	89	....	.....	.....	.....	....	....
		7.4	150	....	.....	.....	.....	....	....
		8.4	159	270.5	5 580	14 450	487	33	56
		9.5	220	....	.....	.....	.....	....	....
		11.7	297	....	.....	.....	.....	....	....
		13.8	380	....	.....	.....	.....	....	....
		16.0	462	....	.....	.....	.....	....	....
		18.0†	780	578.0	10 385	30 800	1 033	61	104
		18.0	....	....	.....	.....	.....	....	....
		20.1	....	641.0	11 405	34 370	1 155	68	115
45		2.1	41	....	.....	.....	.....	....	....
		3.4	87	....	.....	.....	.....	....	....
		4.4	121	97.7	2 682	13 080	574	35	45
		6.2	200	....	.....	.....	.....	....	....
		7.6	269	....	.....	.....	.....	....	....
		9.1†	338	199.9	5 001	26 770	1 170	65	83
		10.4	415	....	.....	.....	.....	....	....
		11.8	496	....	.....	.....	.....	....	....
		13.9	1 000	....	.....	.....	.....	....	....
		14.5	....	....	.....	.....	.....	....	....
		15.9	....	....	.....	.....	.....	....	....
		16.6	....	....	.....	.....	.....	....	....
		21.3	....	469.0	11 056	49 500	2 138	144	183
46		0.3	0	23.7	.....	.....	.....	....	....
		0.8	78	....	.....	.....	.....	....	....
		0.9	104	....	841	18 000	602	12	60
		1.2	186	....	.....	.....	.....	....	....
		1.8	314	....	.....	.....	.....	....	....
		2.3	450	....	.....	.....	.....	....	....
		2.6	560	....	.....	.....	.....	....	....
		3.3	1 000	....	.....	.....	.....	....	....
		3.4†	....	64.9	2 120	49 200	1 648	30	151
		3.9‡	....	72.4	2 360	55 000	1 837	33	168

† Load at first indication of failure. ‡ Ultimate load.

4 and 6) this occurred quite near the yield point of the steel. The bond stress (Column (10)) was as much as 180 lb per sq in. and the shear stress (Column (9)), as much as 102 lb per sq in.

Although the mortar was changed from a straight cement-sand to a cement-lime-sand mixture it seems that the higher stresses obtained were due mostly to the use of stirrups. This would indicate that failure in the case of Items Nos. 1 and 2 would also have been delayed if stirrups had been used. The six beams in this group failed gradually and were removed from the testing machine whole, indicating that a factor of safety can be established on the basis of gradual failure. In testing Item No. 6, the load was applied five times up to 14 000 lb, after which the permanent deflection was 0.036 in. and the elongation of the steel was 0.0006 in. in a gauge length of 8 in. Then the load was increased to 17 000 lb, causing a deflection of 0.387 in. The allowable deflection is 0.4 in.

It is unfortunate that the true widths of the joints were not recorded, because it is of prime importance to know how wide the mortar joint must be to develop sufficient bond. It seems improbable that they were wider than 1 in., which would provide for only $\frac{1}{8}$ in. of mortar between the $\frac{3}{4}$-in. rods and the brick. In other words, the mortar was in contact with the bars only at the top and bottom to all intents and purposes.

Important observations reported were: (*a*) The beams were built of shale bricks with an average compressive strength of 5 953 lb per sq in.; (*b*) the bricks were sprinkled in the pile; (*c*) no brick or mortar was crushed during the test; (*d*) no bottom bricks fell off; and (*e*) a very dry mortar was used.

Items Nos. 7 to 16, Table 1.—Of the tests listed the most comprehensive are those conducted by Messrs. Martin J. Feeney and Raymond W. Miller.[16] The tests consist of four groups of three beams each with two different cross-sections and two span lengths with one size of bar, one mortar mix, and one type of brick, tested to study the action of beams as the load increased. Tests 10, 15, and 16 had clear span lengths of 10 ft, and the distance between concentrated axle loads was 2 ft 8 in. The remainder of the beams had clear span lengths of 5 ft, and the distance between axle loads was 1 ft 8 in. A cement-lime mortar mix of 1:0.15:3 was used throughout, and each of the beams was tested 28 days after it was made. None of the beams reported was reinforced by stirrups. The longitudinal bars consisted of two $\frac{1}{2}$-in. round, deformed bars, all hooked at the ends.

The beams were built of sand struck common clay brick with a compressive strength on the flat side of 4 715 lb per sq in. The reinforcement bars were placed by means of hangers. The mortar in the beams was tested with the following results: Items Nos. 7 to 9, 1 126 lb per sq in.; Items Nos. 10 and 11, 1 138 lb per sq in.; Items Nos. 12 to 14, 1 005 lb per sq in.; and Items Nos. 15 and 16, 1 430 lb. per sq in.

The deflections entered under Item No. 16 require a further word of explanation. The first load of 1 500 lb caused a deflection of 0.003 in. After 24 hours, the load was released and applied again, causing a deflection of 0.005 in. This cycle was repeated twice in 24-hour intervals, with the results shown.

Finally, after six days, the load of 3 750 lb was released, and again applied, to cause a deflection of 0.032 in. The specimen was finally loaded to 13 300 lb, as indicated in Item No. 16, Table 1.

Although the experimenters found a considerable movement upward, of the neutral surface as the load increased, their calculations (for one beam only) showed fiber stresses and values of j and k to be quite close to those of a similarly reinforced concrete beam, and Messrs. Feeney and Miller concluded that reinforced concrete formulas can be used with reasonable assurance in the design of reinforced brick beams. The stresses computed for this group (Items Nos. 7 to 16) show values as high as 1 860 lb per sq in. in the brickwork and 60 000 lb per sq in. in the steel, and shear stresses as high as 84 lb per sq in. at the first crack, with a maximum shear of 154 lb per sq in.

A detailed description of failure is not given for all beams, but the three views of typical failures included in their report show cracks running along

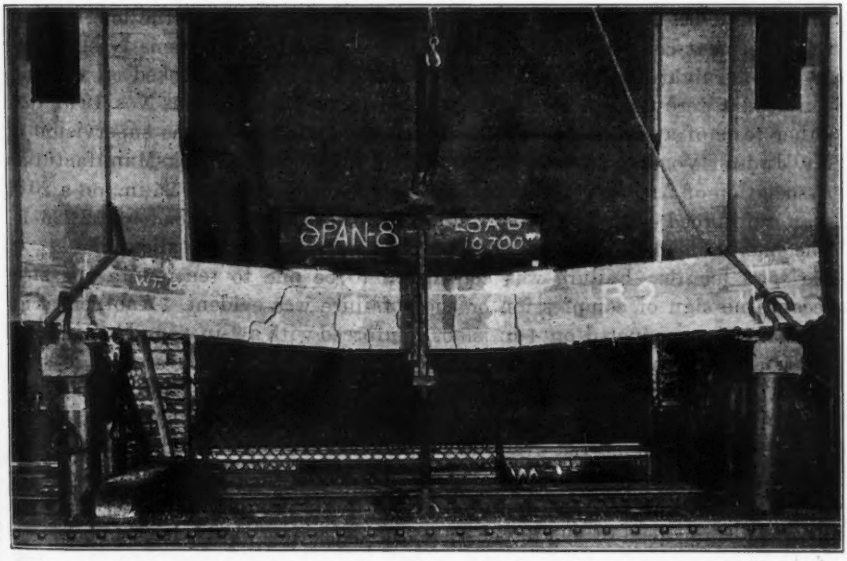


FIG. 1.—BEAM TESTED TO FAILURE BY MESSRS. FEENEY AND MASON (SEE ITEM NO. 11, TABLE 1).

the mortar joints, and, in some cases, through the brick, at various angles and different sections of the beam. One of these beams (see Fig. 1) developed four vertical cracks between the third points, with a fifth crack running diagonally from the third point toward the end of the beam. Another (see Item No. 9, Table 1) had two vertical cracks quite close to, and on either side of, the center line. The third beam (Item No. 14, Table 1) showed a typical shear crack from the support to the third point. The experimenters declared that the primary cause for failure in each case was a weakening of the masonry in tension. However, vertical cracks near the center of the beam indicate either bond or shear failure, and as there was apparently only one

beam showing a typical shear crack, it is logical to assume that in the cases where the writer's calculations show steel stresses to be 50 000 lb per sq in., or more, and bond stresses of 300 lb per sq in., or more, these stresses must have been the causes of failure.

Since the masonry is not expected to take tensile stresses the fact that it did not, can scarcely be given as the cause for failure. The greater bond and shear stresses are reported by Messrs. Feeney and Miller, but with too many variations in materials and construction to determine the cause. It is to be noted, however, that the bars were placed in vertical joints as in Items Nos. 1 to 6, and the mortar was almost the same as in Items Nos. 3 to 6, which appears to be significant.

Items Nos. 17 to 20, Table 1.—The tests recorded as Items Nos. 17 to 20 in Table 1 are largely self-explanatory. The beams were 10 ft 8 in. long, with a clear span of 8 ft. The distance between concentrated loads was 2 ft 8 in., and the mortar mix was 1:0.1:3 for Items Nos. 17 and 18, and 1:1:3 for Items Nos. 19 and 20. The beams had no stirrups and the longitudinal reinforcement consisted of $\frac{1}{4}$-in. round bars. The bars in Items Nos. 17 and 19 were straight and those in Items Nos. 18 and 20 were hooked on the end. Three bars were used in Items Nos. 17 and 18, and five in Items Nos. 19 and 20. Other tests of a similar nature have been conducted under the supervision of Mr. Judson Vogdes, District Engineer for the Common Brick Manufacturers Association of America.[22] These tests include one beam, 8 by 25 in., on a 20-ft span reinforced with five $\frac{3}{8}$-in. round bars that carried a load of 12 226 lb. The maximum deflection was $\frac{9}{16}$ in., or $\frac{1}{8}$ in. less than the allowable $\frac{1}{360}$ of the span length. Failure was considered to be due to tension in the steel because no sign of compression or shear failure was evident. Another series of tests were conducted on 4-in. slabs reinforced with a $\frac{3}{8}$-in. round bar in each joint, and resting on brick beams, 8 ft on center, cantilevered 4 ft over each beam. They were loaded uniformly to gradual failure, under the same cantilever and sustained center-span loads of 995 lb per sq ft. The first crack was a hair crack that appeared over the support at a cantilever load of 2 568 lb, or 350 lb per sq ft, and 648 lb per sq ft on the center span. At 880 lb per sq ft on the center span, a crack appeared across the bottom at the center. At the ultimate center load of 995 lb per sq ft, the deflection was $\frac{7}{32}$ in., or about $\frac{1}{360}$ of the span.

In other tests on the same type of structure, in which the mortar was 1:1:6 instead of 1:0.15:3, the ultimate load was about 825 lb per sq ft. Providing for a factor of safety of 3.5, the safe load for these slabs would be 284 lb. For such a load a reinforced concrete slab with an effective depth of 4.15 in. would be required on the basis of 800 lb per sq in. for compressive stresses in the masonry. The effective depth of the slab tested was $3\frac{1}{2}$ in. Available photographs of these tests, however, indicate an arch action in the loading material which, if true, would modify these results considerably.

Items Nos. 21 to 25, Table 1.—Professor T. G. MacCarthy has reported[23] tests of twenty brick slabs, six brick beams, one mortar slab, and one concrete slab. The mortar slab consisted of the same mix as the mortar used with the

[22] *The Clay Worker*, August, 1931, p. 82.

brick, which was 1:0.15:3. The concrete was a 1:2.6:3.5 mix, with $7\frac{1}{4}$ gal of water. Compared with the brick slab of equal dimensions the reinforced concrete slab deflected 0.585 in. at the ultimate load of 10 485 lb, while the brick slab deflected 0.390 in. at the ultimate load of 10 985 lb. They both failed in shear at about the same stress, 150 lb per sq in. for the brick, and 147 lb per sq in. for the concrete. This is not offered as a typical comparison, but merely as a matter of interest.

The following observations are of interest in connection with Professor MacCarthy's tests: The brick used had a compressive strength flat of 3 850 lb per sq in. Tested on end the compressive strength was 1 850 lb per sq in. The plain bars used for reinforcement had an elastic limit of 40 000 lb per sq in. at an ultimate strength of 73 200 lb per sq in. The specimens were tested at 28 days: 2 by 4-in. cylinders proved to have a compressive strength of 2 600 lb per sq in. All brick was thoroughly wetted before laying.

Items Nos. 21 to 25, Table 1, represent beams having an over-all length of 10 ft and a clear span of 8 ft. The distance between the concentrated loads was 2 ft 8 in. The age at which the beams were tested was 28 days. In Item No. 25 $\frac{1}{4}$-in. round bars, 8 in. on center, were used for stirrups and in Item No. 24 the stirrups were of No. 7 wire, spaced 8 in. on center. The other test specimens had no stirrups. The longitudinal reinforcement bars were neither hooked nor deformed. As indicated in the diagrams in Column (2), Table 1, Item No. 25 was designed with four bars, and the remainder with two bars. In Items Nos. 22 and 25, the bars were square instead of round as in the other tests; $\frac{3}{4}$-in. rods were used in Items Nos. 21 and 24; two of the rods in Item No. 23 were $\frac{1}{2}$ in., and the remainder of the rods were $\frac{5}{8}$ in., in diameter.

The features of Professor MacCarthy's tests were the use of a $\frac{3}{8}$-in. mortar pargeting on the top and bottom of the slab and mortar joints so wide that the thickness between the rod and brick was at least one-half the diameter of the rod. The results obtained were quite satisfactory, but due to the use of the mortar pargeting no comparisons with other tests can be made. The five tests, Items Nos. 21 to 25, are likewise unusual in that a bottom course of mortar was used to embed the rods and a course of mortar for a finish; but it should be noted that the highest stresses were developed in Item No. 23 in which the rods were placed in the vertical mortar joint. The extent to which the mortar courses affected the results cannot be decided, because no check tests were made; but when the results in Table 1 are compared with other tests, it appears that the added care and trouble is not justified.

Items Nos. 26 to 31, Table 1.—Beams tested by Mr. Hugo Filippi, Engineer of the Common Brick Manufacturers Association of America, were constructed under conditions comparable to job practice. The six beams in Items No. 26 to 31, Table 1, were each 11 ft $3\frac{1}{2}$ in. long over all and had a clear span of 10 ft. The distance between the concentrated loads was 3 ft 4 in. Three of the beams, Items Nos. 26, 27, and 28, were made of 1:1:6: mortar, and the remainder of cement-lime mortar in the ratio of 1:0.15:3. They were tested at ages of 23, 26, 28, 22, 25, and 27 days, respectively. These beams had no stirrups and the longitudinal rods comprised three straight bars $\frac{3}{8}$ in. in

diameter, deformed. All the brick were salmons and culls with a compressive strength of 2 626 lb per sq in. They were constructed in freezing weather with no protection, the brick being sprinkled in the pile. The load was applied by piling brick on a platform supported at the third point of the beam. Failure of all beams was caused by shear, and, to quote Mr. Filippi, the beams "ultimately failed in bond between brick and mortar", with the consequent slipping of the bars after the diagonal crack opened. At the first crack, the average shear is calculated to be 25 lb per sq in. and, at ultimate loads, 40 lb per sq in. This early failure in shear limited the brick and steel stresses to values far below those possible, which indicates the necessity for stirrups or bent-up bars. The deflections were regular as the load increased and within the design loads they were much less than the allowable $\frac{1}{360}$ of the span length.

All the beams used in this test were constructed so that there was one course of brick below the steel. It is of interest that in no case did these lower bricks fall off.

Items Nos. 32 to 43, Table 1.—The slabs shown as Items Nos 32 to 43 were also built and tested by Mr. Filippi at the same time as the beams. He reports that in some of the slabs the mortar was frozen, and this explains the small loads carried in some cases. Six of the slabs, Items Nos. 32 to 37, had 5 ft clear span, and the remaining six had clear spans of 8 ft. The mortar mix for six of these slabs, Items Nos. 35, 36, 37, 41, 42, and 43, was 1:0.15:3. For the remaining six, the mortar mix was 1:1:6. The slabs were tested at ages varying from 18 to 24 days. Six straight, plain bars were used for reinforcement in each case. The slabs in Items Nos. 32 to 37 were $\frac{1}{4}$-in. round, and the remainder were $\frac{3}{8}$-in round. As in the beam test by Mr. Filippi all the bricks in the slabs were salmons and culls, with a compressive strength of 2 626 lb per sq in. All the slabs were built in freezing weather, with no protection.

In general, the results were as might be expected and indicate that, for spans of 5 ft, a $2\frac{1}{4}$-in. brick slab is safe for loads up to 50 lb per sq ft, and that $3\frac{3}{4}$-in. slabs can be designed to carry 60 lb per sq ft on spans of 8 ft, with a factor of safety of from 5 to 6. In all but two cases (Items Nos 39 and 42), failure was gradual, with regular increases in deflection as the load increased. Two of the slabs were loaded to three times the designed amount and when it was removed, full recovery of deflection was made, showing once more the capacity of this type of construction to recover its deflection. Fig. 2 shows Item No. 43 during the test, carrying 397 lb per sq ft.

Items Nos. 44, 45, and 46, Table 1.—These specimens were built and tested by the writer under the inspection of the Building Department of the New York Central Railroad Company. It was the intention to build them under conditions secured, and with the speed usually obtained, on a contract job. However, these conditions were exaggerated in the case of Item No. 44 when sand for the last batch of mortar had to be scraped from the ground, resulting in a dirty mortar which was used in the top course at the center of the beam. This no doubt contributed to the early failure in compression obtained on this beam.

The two beams had 16-ft and 11-ft spans, respectively, and the slab an 8-ft span. The mortar was 1:0.25:3 cement lime; and the brick, common clay with an average compressive strength of 3 044 lb per sq in. All work was done in the open, and wetted for 10 days. Testing was done at the age of 30

FIG. 2.—SLAB ONLY ONE BRICK THICK SUPPORTING 397 LB PER SQ FT; LIVE LOAD OF 9000 LB (SEE ITEM NO. 43, TABLE 1)

days with improvised platforms resting at the third points on all items. Deformed bars with straight hooks and double $\frac{1}{4}$-in. stirrups were used. Bending was done at the site. Item No. 44 was built in place; Items Nos. 45 and 46 were built on the ground and lifted to the supports on the day of the tests. In doing this several courses were broken from the side of the slab, thus reducing its width by 8 in. to 30 in.

Both beams gave away suddenly in the manner of a breaking stick at loads equal to 2.4 (Item No. 44) and 4.8 (Item No. 45) times the designed loads. The first crack and allowable deflection were at loads slightly greater than twice the designed load in each case. Although the early failure in compression limited the development of shear stresses to 144 lb per sq in. (Item No. 45), still the absence of any indication of shear cracks shows that this stress can be taken care of, even though mortar joints pass completely through the beam every 8 in. as in these items. The tension in the steel (Item No. 45) was 49 500 lb per sq in. at ultimate load. Considering

the dead load also, this stress is computed as 64 500 lb per sq in. Similar increases apply to all the other beams.

These failures were unusual, in that the buckling action at the center caused the top course to pull away for a length of 30 in., causing a complete destruction of the section directly beneath it, although the remainder of the beam was intact.

The slab (Item No. 46) failed gradually. The test was discontinued because of excessive deflection at a load equal to 4.4 times the design load of 60 lb per sq ft, or 900 lb on the basis of third-point loading. After the load was removed five men stood on the slab, and it seemed as stable as ever even if greatly bowed.

The writer understands that other tests are being conducted which indicate that the value of n for reinforced brick masonry should be greater than that assumed herein. However, as nominal variations in the value of n do not materially affect the shear stresses, it is not of great importance, as a review of the stresses developed by the masonry in compression and the known ability of steel to take high tensile stresses give assurance that there is an ample factor of safety even if the exact distribution of stresses is not known.

If future investigations indicate that $n = 30$, the values given herein for the brick stresses will be reduced by about 16%, the steel stresses increased by about 4%, and the shear stresses decreased 4 per cent. When working stresses are based on a factor of safety of 3, or more, these possible errors are unimportant.

Several other tests have been conducted, but due to the incomplete reports available, no definite conclusions can be drawn from them. For instance, Major Lent tested $2\frac{1}{4}$-in. and $3\frac{3}{4}$-in. slabs reinforced with light-weight, expanded metal, diamond mesh. That is, the bricks were laid in a bed of mortar on the mesh and tested on a 4-ft span, which required a load of 520 lb per sq ft to produce a deflection of $\frac{1}{8}$ in., which is $\frac{1}{360}$ of the span length, and made practically a full recovery when the load was removed.

Of course, such variables as the strength of brick and mortar have considerable effect on the capacity of the beams and slabs, but due to the wide variation in all elements of the structures tested in the experiments discussed herein, and to the fact that the tests in general were not designed to determine single characteristics, it is difficult to draw conclusions as to the extent of their influence.

Reviewing Table 1, therefore, it is found that: (1) Deflections in reinforced brick masonry beams and slabs are consistent and small within working loads (see Fig. 3) (usually they are much less than the allowable deflection of $\frac{1}{360}$ of the span length); (2) upon removal of the load, recovery of deflection, sometimes 100%, is possible; (3) granting the assumptions on which the calculations are based, rods inserted in mortar joints will develop bond stresses as high as 384 lb per sq in. (see Item No. 8), with an average of 229 lb per sq in. when rods are placed in vertical joints (see Table 2); (4) shear stresses are affected by the manner in which the bricks are placed, and by proper design can be developed to as high as 154 lb per sq in. (Item No. 8) without shear reinforcement; (5) the introduction of stirrups increases the

beam's resistance to shear (see Table 2); (6) when properly designed, the tensile strength of longitudinal reinforcement can be fully developed; and (7) the compressive strength of the masonry in beam action is greater than

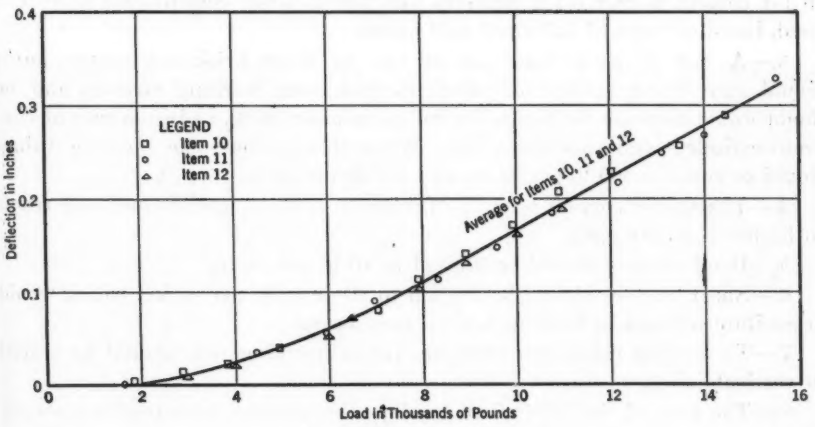


FIG. 3.

the compressive strength of wall panels in direct compression. Table 2 does not take into consideration the manner of failure. Therefore, one or the other stress is always less than that possible; thus, the values listed offer conservative values.

TABLE 2—AVERAGE AND MINIMUM UNIT BOND STRESS AND SHEARS

Item No.	Description	Unit Stress, in Pounds per Square Inch					
		Bond			Shear		
		Number of beams averaged	Stress		Number of beams averaged	Stress	
			Average	Minimum		Average	Minimum
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	All beams listed at first indication of failure	31	124	51	31	61	23
2	All beams listed at ultimate load	32	187	65	32	88	29
3	Rods in vertical joints at ultimate load	18	229	82	.....	....	.....
4	Rods in horizontal joints at ultimate load	9	116	65	.....	....	.....
5	Rods in mortar at ultimate load	4	164	98	.....	....	.....
6	All beams without stirrups at first indication of failure	.....	....	....	24	53	23
7	All beams without stirrups at ultimate load	.....	....	....	25	83	29
8	All beams with stirrups at first indication of failure	...	....	....	7	90	61
9	All beams at ultimate load with stirrups	.....	....	....	7	108	68

CONCLUSIONS

In view of the evidence submitted in this paper, the writer presents twelve conclusions for discussion, as follows:

1.—The assumptions accepted in the design of reinforced concrete structures can be used in the design of reinforced brick masonry items similar to those discussed herein.

2.—The recommended working stress of 0.4 f'_c given by the Joint Committee on Standard Specifications for Reinforced Concrete can be safely allowed on reinforced brick masonry. This would allow approximately 750 lb for cement-mortar brick masonry and 500 lb when cement-lime mortar is used, based on tests of full-sized wall panels.

3.—A test of piers made up of two or three brick-and-mortar joints would give better results on which to base such working stresses and no doubt would increase the foregoing values by about 10%, as in the case of concrete cylinders and concrete walls. When this is done, the limiting values would be roughly 800 lb per sq in. and 550 lb per sq in.

4.—The use of smaller bars ($\frac{1}{2}$-in. round) is more feasible and will result in higher bond strength.

5.—Bond stresses should be limited to 80 lb per sq in.

6.—Shear stresses should be limited to 25 or 30 lb per sq in., which would necessitate stirrups or bent-up bars in most beams.

7.—To develop maximum strength, the reinforcing rods should be placed in vertical joints.

8.—The face of the brick that develops the greatest compressive strength should be placed normal to the line of compressive stress in the beam.

9.—The value of the ratio, $\frac{E_s}{E_b}$, is somewhere between 20 and 30.

10.—The mortar should consist of 1 part cement, 0.25 part lime, and 3 parts sand, by volume.

11.—Builders should adhere rigidly to all the requirements of good brick masonry; that is, the proper wetting of the brick and complete filling of joints.

12.—Further well-directed study and testing will undoubtedly justify an increase in the allowable stresses.

Given these conditions, this type of construction can be used with safety when designed by the formulas given in the report of the Joint Committee on Standard Specifications for Reinforced Concrete. The development of any system of construction is governed by economic considerations, and there are certain handicaps to the use of this type, such as (*a*) the necessity of placing rods in mortar joints, which will sometimes result in an excess amount of steel; (*b*) the need for considerable hand labor, both of which, (*a*) and (*b*), are expensive; and (*c*) the difficulty of securing water-tight joints in exposed work to prevent the corrosion of the steel. There are undoubted advantages, however, such as the elimination of all vertical form work, the flexibility of operations permitted in a small working space, the finished appearance, and the invisibility of cracks—which will often more than balance the disadvantages for such construction as retaining walls, circular tanks (see Fig. 4), piers, and bridge bents, and floor-slabs when architectural considerations require them.

Other possibilities that suggest themselves are stadiums, foundation walls, tunnels, etc., where formwork would be intricate and expensive, or concrete

difficult to place. Reinforced brick curtain walls have been utilized[23] in a six-story steel frame structure with 8-in. walls, in San Francisco, Calif. This use of reinforced brick masonry would appear to be logical in areas subject to earthquakes. Then, of course, there are situations in which concrete and steel are not available, when a knowledge of this method of construction

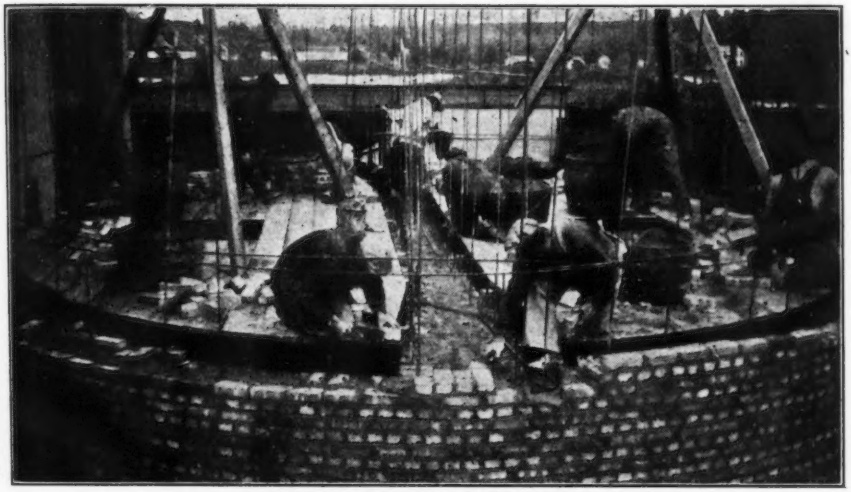


FIG. 4.—STORAGE BINS AT WENDRON, ILL., SHOWING REINFORCEMENT.

would be of great value. The weight of such construction is about 20% less than that of concrete, which would have considerable effect in a multiple-story structure, and the dead load can be further decreased by the elimination of ceiling and wall plaster in buildings where the structural members will become decorative when made of brick. The use of brick for interior finishes is found in ancient as well as extremely modern buildings and the variation in color obtainable in ordinary brick allows the designer great scope for decorative effects.

If developments indicate that specially shaped brick would be advantageous, such as slotted or grooved brick, the brick industry will no doubt respond as it did when radial brick for chimneys and feather brick for arches were deemed necessary.

ACKNOWLEDGMENTS

The writer is indebted to Carl T. Schwarze, M. Am. Soc. C. E., for offering helpful suggestions after reading this paper. The paper was prepared from data secured while the writer was employed by the New York and New Jersey Common Brick Association, and acknowledgment is due that source.

[23] *Brick*, Vol. 43, No. 6, 1913.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

THE PLAN OF BOSTON, MASSACHUSETTS: A CAPITAL CITY[1]

BY ARTHUR C. COMEY,[2] M. AM. SOC. C. E.

SYNOPSIS

State capitals, by example, inspire other cities in planning. The State functions may dominate a small city, or they may merely furnish the basis for a fine architectural treatment as in Boston, Mass., the largest State capital. Boston's plan relates to its commercial and other municipal activities as a metropolis. It also shows the effect of its long history of 300 years. It still has many early architectural gems.

At first, the streets were narrow and crooked. Later, checker-board streets were laid out in new territory and on filled-in mud-flats, thus, destroying Boston's peninsular shape. Some streets were made wide only to become narrow again upon entering the still more recent suburban districts, many of which became a hodge-podge of narrow streets leading nowhere.

Since about 1890 broad planning has received more and more attention. After a long series of haphazard street widenings, an official Planning Board prepared a major street plan in 1930. This Board had previously zoned the city and has become recognized as the central authority on planning matters.

Waterways, long ignored, have become the centers of beautiful parkways. Since the creation of the Board, Boston's port development has followed a general plan. For some time railroads have been fixed, with two consolidated terminals. Rapid transit, beginning with the first American subway, taps all inner suburban areas. Boston's park system has led the way; likewise, the City established the first municipal playground in the United States.

Boston has no civic center, but the State House faces the great central Common. It is protected by a low height limit imposed on adjoining build-

NOTE.—Discussion on this paper will close in August, 1933, *Proceedings*.

[1] Paper presented at the meeting of the City Planning Division, Boston, Mass., October 10, 1929.

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ings. Boston's pioneer control of building heights by districts was followed by comprehensive zoning. By statute dated March 19, 1928, pyramidal buildings with the same volume as a solid 155-ft building are permitted.

As Metropolitan Boston includes thirty-nine or more municipalities there was little regard for the greater city's future prior to 1929. Only water supply, sewage disposal, and the larger park features were under the control of Metropolitan Boards, which were merged into a single commission in 1920. The State exercises much detailed authority over the area. It has set up a Division of Metropolitan Planning, which has prepared a thoroughfare and transit plan as a guide for the Legislature and the individual communities. The State Department of Public Works also has prepared plans both within and without the City of Boston. While there is overlapping, there is no metropolitan planning of zoning, and little co-ordination of many other factors in a regional plan. The Division of Metropolitan Planning probably should be in charge of such a plan covering all problems affecting more than one municipality.

The chief justification of a paper such as this lies in the inspiration and guidance that the capital city may exert through its example on other cities within the State. The prototype in this is the Capital of the United States, which, itself originally well planned and now receiving renewed attention as to its planning, undoubtedly influences profoundly the quality and extent of planning in cities throughout the country. State capitals can seldom if ever approach in effect that of the Federal Capital, but they may, and do, in many instances, lead the State in city planning ideals.

Boston, the Largest Capital

A second factor worth comparing in capital cities is the relation of the State's administrative function to the city's physical plan. This also varies exceedingly in importance, largely according to the magnitude of each city's remaining functions. Denver, Colo., presents what may be considered the arch-type.[8] Another type is that represented by Olympia, Wash., in which the population is small and the capital is devoted almost entirely to State administrative activities. In such a capital as Boston—the largest of them all — the presence of the State Government is only an incident. To be sure, the one large capitol building is the crowning jewel of an architectural composition in one part of the city (see Fig. 1), but otherwise it exercises practically no effect upon the city plan. To satisfy oneself that this is so, it is only necessary to compare the elements of Boston's city plan with the same elements in any other great center of population. The city is one of the small group of world metropolises. Its effect upon the surrounding region

[8] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1312.

is due to its size, to its being the commercial center of this region, to its being a seaport, and to the industrial, educational, and other activities that have been attracted to it. Had the capitol been placed, as in many States, in

FIG. 1—THE STATE HOUSE ON BEACON HILL, BOSTON, MASS.

some small more central city, Boston's plan would have shown few or no differences. Therefore, this study is directed at Boston as a metropolitan commercial city and not as a capital.

ORIGIN AND EARLY DEVELOPMENT

Boston differs also from most other capitals, particularly those of the West, in being one of the oldest cities in the United States, and thus exhibiting much more of an historical character in its plan. The tercentenary of the city was celebrated in 1930. Never planned in any large degree, it has grown steadily from the small Seventeenth Century Colonial village and carries in its layout many visible evidences of each successive era. The city's history is a subject that is worthy of extended study by the city planner. The first settlement was across the Charles River from Boston proper, at Charlestown, which was laid out by an English engineer, Thomas Graves, with 2-acre lots for each inhabitant. In Colonial days, Boston was situated between and upon three hills on a peninsula with a long, narrow neck, easily defended from the Indians. It was flanked by salt marshes, but jutted into the deep water of a fine harbor. On one side land was reserved for a common ground, and the Boston Common still remains as the beginning of the city's park

system. In this first era the streets were nearly all narrow and often crooked, serving the immediate need only. Many gems of architecture, both public and private, remain from this and the immediately succeeding periods, but few of the other municipal activities have left a permanent stamp on the present city plan.

However, at an early date, mud-flats and other areas began to be filled and this work has continued spasmodically, resulting in great changes in the topography. First, certain coves were obliterated; then large areas along the principal estuaries were filled until, to-day, Boston is no longer a peninsula; it is even losing its shape as a broad point. Each of these filled areas offered a comparatively clean slate for a new street plan and—as in the case of practically every other planned city from the earliest Egyptian and even lacustrine times practically to the present century—these fragments were planned on a checker-board system, or as close to it as their marginal limitations permitted.

At this same period, Boston began to extend its city limits, absorbing neighboring towns and intervening land in several directions although not in all. Where growth was slow these towns expanded in a haphazard fashion, but certain large areas that came into the market about 100 years ago shared in the world-wide attempt at a grander scale of city building. These areas were laid out with straight streets (some of them very broad) again plotted in the checker-board plan, in some cases practically regardless of unfavorable topography. Thus, the street plans of East Boston, South Boston, the South End so-called, lying between the old Boston proper and Roxbury, and, later, the Back Bay District were added, with a resulting ease of movement of traffic, the benefits of which are being reaped to-day.

During this early Victorian period even the pre-existing roads were widened in some cases as they passed through the newly planned belt, so that Washington Street, for example, broadens from 50 ft to 100 ft, and more, in width in the South End only to become narrow again at its entrance to Roxbury.

What may be considered the first city plan for Boston was developed from 1843 to 1845 by Robert Fleming Gourlay and submitted by him to the authorities as "Studies to Illustrate the Science of City Building"; but although he anticipated later needs with surprising accuracy, his plans apparently fell on deaf ears.

Later, the generous ideals of the early developers were abandoned, and many miles of streets were added to the city plan with little or no thought for community needs, so that the outer sections of the city are for the most part a hodge-podge of unrelated fragments—narrow streets leading nowhere. The pre-existing main streets usually passed through these suburbs with their original narrow country-road widths. However, here and there, a better plan was followed, already a few high-grade sub-divisions were being platted, and since the land was cheap in the outer circle, some of the thoroughfares again were widened. Thus, in passing outward, on Blue Hill Avenue after more than a mile of 60-ft street, the width changes to 120 ft from that point to the city line.

Development in Recent Years

Since about 1890 two special influences have been brought to bear on the street plan. The park system became important. To-day, the parkways laid out at that time are practically integral portions of the traffic network, although happily restricted to passenger travel.[4] Finally, broad planning, at least so far as related to streets, was attempted through a Board of Survey. Much of the unplatted portion of Boston was laid out on a general plan of wider streets, with a reasonable number of through connections and some relation to topography. Although the invocation of the police power to assure the completion of this plan was soon declared unconstitutional, many portions of the layout were accepted and now exist on the ground. While mechanical in some respects and not as fully related to the needs of the city as a whole as modern city planning would dictate, this street planning was a long step ahead and its methods have never been entirely abandoned by the Board's successor, the uneuphoniously but accurately styled "Street Laying-Out Department" (see Fig. 2).

At an early date Boston began widening its old streets and cutting through new ones to facilitate circulation. As a result of the fire of 1872 a large portion of the older city received wider streets, but with few changes in the general arrangement. In 1929, it was estimated that $7 000 000 had been spent on widening and straightening the streets to that time. At first, these improvements were successive, isolated projects. Several streets have been widened two or even three times, while other proposals have languished for many years before being acted upon. More recently temporary unofficial and official agencies have produced plans relating several improvements into a single plan for general relief. Usually, only a fraction of such projects was actually finished.

Work of Planning Board

Finally, following a compulsory State law sponsored by the Massachusetts Homestead Commission, a permanent official Planning Board was established in 1914, although without any powers other than those of advice. At first, ill-understood, it has gained ground gradually until to-day (1932), it is generally recognized as the central authority on planning matters for the city. At the outset the Board had to content itself perforce with promoting specific projects that appeared certain to be of value in any ultimate scheme; but, strengthened by its success in such undertakings as the Stuart Street and the Cambridge Street widening, it successfully launched a general zoning plan in 1924. Since then the Board has accumulated data and, with the aid of a special staff, it published in 1930 a comprehensive major street plan relating all street improvements to one another and to the city's ultimate needs (see Fig. 3).

Drainage Development

Other factors in Boston's plan became important long after its street plan was far advanced. The development of the system of land drainage,

[4] *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2491.

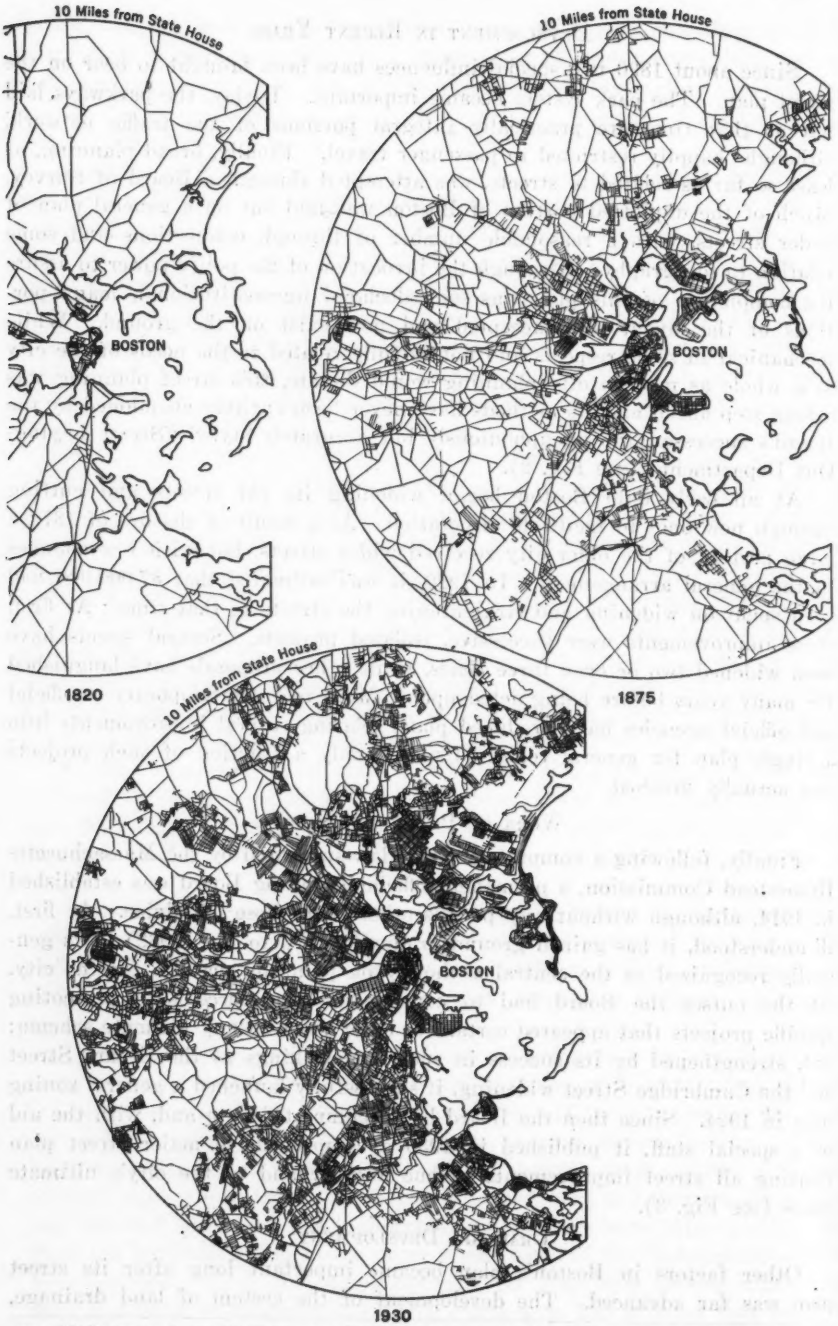

FIG. 2.—GROWTH OF BOSTON, MASS., AND VICINITY AS INDICATED BY EXTENSION AND DEVELOPMENT OF THE HIGHWAY SYSTEM.

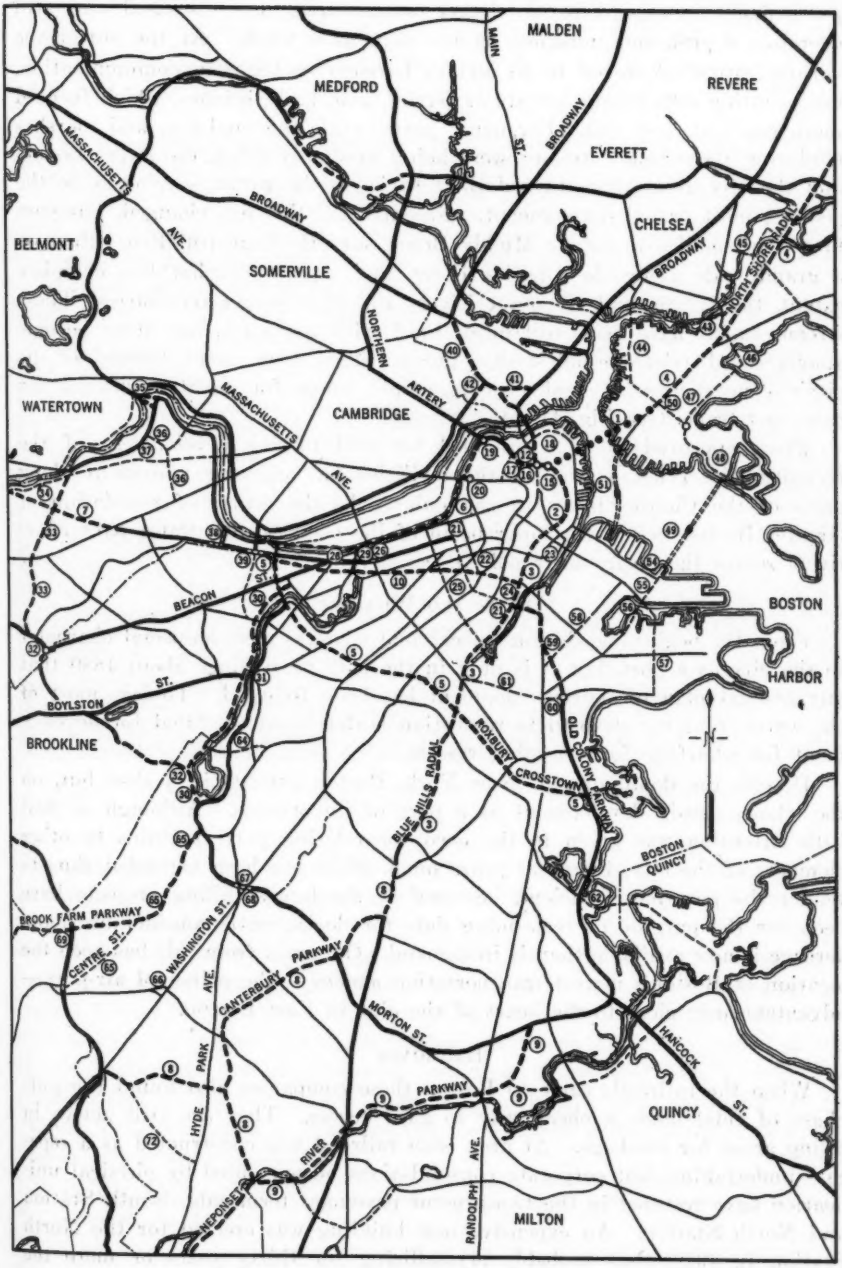

FIG. 3.—THOROUGHFARE PLAN, BOSTON, MASS.

particularly as related to the larger watercourses, has exercised and still exercises a profound influence on the plan as a whole. At the outset the various estuaries proved to be serious barriers to highway communication, necessitating either long detours or ferries, and, later, bridges, which focused main roads at each end. For many years, while the mud-flats and marshes bordering these small streams were being gradually filled, the street system and the city as a whole turned their backs to the water, leaving it to the poorest, most casual development. Recently, all this has changed, however, first on a small scale, along Muddy Brook, now the beautiful Fens; then on a grand scale along the Charles River, and, to a somewhat less complete extent, the Mystic and Neponset Rivers and their larger tributaries. These stream basins have been reclaimed, lined with parkways, and have become among the district's proudest show-pieces. They form great "lungs" to the city's congested areas and provide ample space for great parkways for passenger traffic (see Fig. 4).

Plans approved in 1929 provided for still further development of the recreational service, as well as the facilities for vehicular movement along parts of the Charles River in particular. As the center of population of Greater Boston lies in the lower basin of the Charles, this water park effectively serves the entire metropolitan community.

Docks and Wharves

From the beginning Boston's docks and wharves gave a special character to the city as a port, but it is only in the fifty years since about 1880 that any general plan of port development has been followed. To-day, most of the water-front not devoted to recreation is developed or slated for development for wharfage for sea-going vessels.

Despite the dominance of New York, Boston persists in a slow but, on the whole, steady development as a port of importance. Although at first little attention was given to the need for relating port facilities to other elements of the city, in recent years much effort has been expended thus to render the port more efficient. Several of the largest filling projects have been for the purpose of reclaiming flats for docks, until considerably more acreage is now available than is in demand. One gain from this has been the location of Boston's newest transportation agency — the principal air-port — advantageously close to the heart of the city in East Boston.

Railroads

When the railroads came to Boston these companies also found the purchase of tidal lands a cheap way to gain access. They are still active in filling areas for trackage. At first, each railroad was constructed as a separate undertaking, but corporate consolidations accompanied by physical unification have resulted in the two present passenger terminals: South Station and North Station. An expensive new building was erected for the North Station in 1928, thus probably crystallizing for thirty years or more the present none-too-satisfactory arrangement, particularly as regards suburban commuting travel.

At first, street cars simply fitted into the crooked street system, but this produced so much congestion that in 1898 the first American subway was built. Since that time all surface cars have been removed from the center of the city, and rapid transit lines have been constructed to tap all the inner

FIG. 4.—DOWN-TOWN BOSTON AND THE HARBOR, WITH CAMBRIDGE, MASS., BEYOND.

suburban areas. The detrimental effect of elevated structures soon dictated the abandonment of that system in favor of tunnels and, in the latest instance, in order to economize expense, the open-cut system, utilizing existing railroad rights of way.

The development of sanitary sewerage[5] and water supply[6] has had comparatively little effect on the general plan of Boston.[5] Pipe lines, large and small, have followed streets rather than streets following the best alignment for pipes. To be sure, water supply reservoirs are important landscape features of the park system in several instances. Aqueducts have usually been buried, and the land often left in private hands, although one or two of the newest constitute unused opportunities for through walking routes extending many miles into the country.

[5] *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2509.
[6] *Loc. cit.*, p. 2496.

Park System

As distinguished from circulatory services, whether of people or of goods, land uses in Boston, which are really the end product of a city plan, may be divided into two main classes—public and private. In so far as they are important in the city plan, public lands may be divided into lands for recreational service and lands for administrative service. Boston led the way in a connected series of parks, forming an actual system and, later, two interconnected systems—an inner city system and an outer metropolitan system. Most of these systems were planned before the days of automobiles when the present service to traffic rendered by the parkways was unforeseen. The parks themselves have continued to increase in usefulness for recreation, although the outlying ones are still in large degree what their generic name strictly means—"reservations" for the future use of a population that may either need them more or appreciate them more.

Boston was also a pioneer in supplying playgrounds, establishing the first municipal one in the country—the Charlesbank—in 1889. Others were added from time to time, and beginning about 1920, a vigorus attempt was made to place one within easy reach of every section of the city, even to the point of tearing down blocks of tenement houses to make the necessary space. Likewise, most of the available salt-water beaches, as well as a few of the inland ponds, have been developed for bathing, so that if present progress is continued steadily Boston will never be behind in providing recreation.

While the playgrounds are often close to schools, they are not, as a rule, a part of the school yard. Schools, therefore, have exercised less effect on the city plan as a whole, although certain large high schools, often constituting a group of buildings, vie with the colleges and other institutions sprinkled over the city in presenting effective civic architecture.

The Capitol and Its Site

Administrative buildings, in some instances, set a high standard for civic art. In Boston, the Capitol, or State House, stands pre-eminent on Beacon Hill (see Fig. 1). Threatening tall buildings have been kept away from it by condemning air rights above certain heights. The main axis of the building crosses the great central open space of Boston Common. Boston's charm is much enhanced by casual views of the dome of the State House as a dominant feature in the skyline across Charles River Basin, or of the building as a terminus of the street vista, as, for example, up Tremont Street from its intersection with Stuart Street. However, no other State or municipal buildings are grouped with the State House, and Boston has no civic center. Its City Hall, Court House, Post Office, and other important buildings are each tucked away on some narrow street or all too-small "square." Those few State functions not housed in the State House are located in such remote places as the Commonwealth Pier in South Boston and in scattered rented quarters.

Zoning

Boston early regulated the height of all buildings. A State law prohibits any building higher than 125 ft in any Massachusetts city, except Boston. In 1904, Boston really anticipated the zoning movement by dividing the city into two districts, with a lower height limit in the second. In 1923, the height limit in the less restricted district was raised to 155 ft, but with set-backs, if necessary, so that no part would exceed two and one-half times the distance to the farther side of the street.

Following a Constitutional Amendment and Enabling Act, Boston was zoned in 1924 under the writer's direction. Use districts were classified as Single Residence, General Residence, Local Business, General Business, Industrial, and Unrestricted; and bulk districts were classified according to height, as 35-ft, 40-ft, 65-ft, 80-ft, and 155-ft districts, each with appropriate requirements for open spaces. Subsequently, in 1928, the writer participated as Consultant in changing the maximum height limit to permit taller buildings within a pyramid-shaped prism starting 125 ft above the street, provided their volume did not exceed that previously permitted.

Administrative Problems in Planning

Two administrative factors have exercised a vital effect on the plan of Metropolitan Boston—its relation to the State and its separation into thirty-nine municipalities.[7] Due to the latter, progress in a unified development has been slow. Each municipality has worked out its own salvation, often with no regard to the others or to its own future as an integral part of a metropolis. Thus, radial highways are numerous, although narrow, while cross-town or circumferential connecting highways are either lacking or are imperfect, with numerous missing links. In the six years beginning with 1889, those services most imperatively demanding consolidation were so treated under metropolitan commissions. Twin sewage disposal systems were built to receive sewage from each municipality and carry it to sea. Most of the cities and towns join in a Metropolitan Water Supply System, the water being delivered to each municipality for local distribution. When a need for outlying parks, reservations, and beaches was appreciated another Metropolitan Commission was organized to secure these and, later, to connect them with parkways. Although these three services were put under a single head in 1920, they were each designed as a separate project and also without much relation to the other elements of the city plan. In 1929, the organization of a Metropolitan Transportation District was initiated.[8]

Owing to the preponderating importance of Boston and the District in the State, another factor in its development is still very much in evidence, namely, the State Government itself. The Legislature dictates many of the details of local activity, particularly whenever the project is of such size as to be outside the ordinary debt limit of the municipality. Thus, Boston's building code, zoning code, and many of its major street improvements, as well as practically all metropolitan projects, are controlled by special acts

[7] *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2485.
[8] *Loc. cit.*, p. 2507.

of the Legislature. After years of piecemeal work a Division of Metropolitan Planning was set up, which advises the Legislature on various large thoroughfare and transit projects for immediate relief both inside the City of Boston and elsewhere. This Division has prepared a comprehensive metropolitan thoroughfare and transit plan both as a guide for the Legislature and for individual communities. In 1929, another agency stepped into the field, the State Department of Public Works, with a plan for a radial highway northeast from the East Boston Tunnel, through East Boston, to existing highways in Revere, Mass.

City Planning Needs

One result of this multiplicity of planning agencies, with overlapping and sometimes conflicting fields of activity, has been to stultify local planning work except in Boston itself. Moreover, it has resulted in more than the usual confusion and expenditure of human energy in getting anything done. Again, as no one body is yet charged with the duty of preparing a comprehensive regional plan, only certain phases are covered at all. The planning of highways and transit receives close attention by the Division of Metropolitan Planning. The park system receives attention only in so far as it happens to relate to the highway system and to present parks and parkways. Metropolitan zoning of the use and bulk of buildings receives no attention although it is needed very much indeed.

Probably the most effective way to get ahead will be to charge the Division of Metropolitan Planning with the duty of preparing and keeping up to date a comprehensive Metropolitan Regional Plan and of constantly presenting this plan before the Legislature and to local planning agencies. Naturally, it would confine itself to problems affecting more than one municipality and would stimulate the local planning boards to develop and promote comprehensive city and town plans for their respective areas. The progress of Boston in developing according to a plan has been great. Its opportunity for further similar progress to meet current urban needs is even greater.

Acknowledgments

The writer wishes to record his appreciation to the City Planning Board of Boston, Mass., for courtesies in supplying copy for several of the illustrations in this paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

ENGINEERING FEATURES OF THE ILLINOIS WATERWAY

Discussion

BY WALTER M. SMITH, M. AM. SOC. C. E.

WALTER M. SMITH,[21] M. AM. SOC. C. E. (by letter).[21a]—Since the publication of this paper the Illinois Waterway has been practically finished, except for the completion of some of the bridges.

Owing to the tremendous increase in costs from the time the bond issue was voted for the construction and the time of construction, it was realized, in 1929, that the funds provided by the bond issue would not be sufficient. There were two possible ways to complete the Waterway. The first was for the State of Illinois to vote an additional bond issue. As the Illinois Constitution prohibits the State from lending its credit to the construction of railways or waterways, this would require an amendment. The Illinois Constitution also prohibits more than one amendment being voted on at any election, and there is always some important amendment to be voted on at each election. It was evident that this method would take years for completion, if successful, and there was a strong probability that it would not be successful.

The second method was to ask the Federal Government to furnish the money and complete the Waterway. It was impossible to turn the Waterway over to the possession of the United States Government because it belongs to the people of Illinois and can only be surrendered by a popular vote and very probably it would require another amendment to the Constitution to allow this vote to be taken. However, the President recommended, and Congress provided, the funds to complete the Waterway, except the bridges crossing it. The State, therefore, stopped work in 1930 to retain sufficient funds to construct the bridges.

NOTE.—The paper by Walter M. Smith, M. Am. Soc. C. E., was presented at the meeting of the Waterways Division, Milwaukee, Wis., July 11, 1929, and published in October, 1931, *Proceedings*. Discussion on the paper has appeared in *Proceedings* as follows: February, 1932, by Messrs. A. W. Sargent and E. G. Walker; and March, 1932, by Messrs. David A. Molitor, and Charles A. D. Young.

[21] Asst. Chf. Engr., Div. of Waterways, State of Illinois, Chicago, Ill.

[21a] Received by the Secretary January 9, 1933.

The United States War Department began work on the project in February, 1931, the State concentrating its work on the bridges. At this time the lock and dam at Starved Rock, the Marseilles Lock, and the Lockport Lock were practically completed, except for the installation of equipment for electrical operation of lock-gates and valves. Brandon Road Lock was finished, except the installation of operating machinery, and the dam and walls through Joliet were about 75% completed. Marseilles Canal was about 90% completed. Dresden Island Lock was about 30% completed, but only a portion of the excavation had been done for the dam.

Marseilles Dam had been designed but work had not been started on it. All metal work had been provided for Dresden Island Lock and Dam as well as the machinery for operating the gates and valves at Dresden Island and Brandon Road Locks.

The Des Plaines and Illinois River Valleys have abundant banks of gravel of a fine quality for concrete. In many of them the gravel does not need to be washed or screened to size. Concrete, with bank-run gravel, was used in all the structures built by the State, because it had been found satisfactory for massive works and was much cheaper than the washed and screened material. The Engineers of the War Department were not accustomed to the use of bank-run gravel in concrete so when they took over the Waterway, they very naturally wished to satisfy themselves as to the quality of the concrete in the structures built by the State Engineers. Therefore, they had many cores, 6 in. in diameter, cut to a depth of 10 ft in the Lockport, Brandon Road, Marseilles, and Starved Rock Locks. Sample pieces, 12 in. long, were then cut from these cores and tested, generally from two to four test specimens from each core. Wherever there were places in the cores that appeared bad, they were included in the test specimens because the engineers wished to know the worst. In many places one stone would occupy more than one-half the area of the test cylinder. The results are given in Table 2. It is to be noted that from one to five test pieces were cut from each core.

TABLE 2.—SUMMARY OF CORE TESTS OF CONCRETE

Lock	Number of cores	Number of specimens	Breaking Strength, in Pounds per Square Inch		
			Minimum	Mean	Maximum
Lockport	5	19	3 117	5 746	7 874
Brandon Road	4	15	3 915	5 228	6 913
Marseilles	3	11	2 779	4 975	7 666
Starved Rock	4	15	3 510	5 038	7 689
All tests	16	60	2 779	5 306	7 874

This concrete generally contained from $4\frac{1}{2}$ to 5 bags of cement per cu yd, except for the upper part of the lock-walls where the water rises and falls. In this part of the work generally about 6 bags of cement were used per cu yd of concrete, in order to obtain greater density and to prevent spalling of the face from frost action.

Before turning the Waterway over to the War Department Engineers, tests were made on the operation of the gates and valves at Lockport, Marseilles, and Starved Rock Locks.

The specifications called for operating each pair of miter-gates through fifty complete cycles continuously, allowing 2 min each for opening and closing and a $\frac{1}{2}$-min allowance at each end of each swing. This would require each pair of gates to be operated fifty times in 4 hr and 10 min. The lower gates operated under service conditions, but the upper gates had to be operated "in the dry." The lock-gates at Starved Rock and Lockport were swung only five times in order to avoid injuring the pintles. At Lockport, the upper gates were required to open or close in 2 min. No specific test was prescribed. These units are vertical lift-gates, only the lower gates being of the miter type. The leaves are 59 ft high and weigh about 320 tons each. The results of the tests of the miter-gates are shown in Table 3.

TABLE 3.—TESTS OF MITER-GATES

Location	Lower Gates (150 Cycles)					Upper Gates		
	Average time of opening, in seconds	Average time of closing, in seconds	Average time lost, in seconds	Total time elapsed, in hours	Total time allowed, in hours	Average time of opening, in seconds	Average time of closing, in seconds	Average time lost, in seconds
Lock-Gates								
Starved Rock	118	119	0.45	3.75	4.17	105	105	.....
Marseilles	108	108	0.30	3.25	4.17	109	109	0.0
Lockport	109 to 114	109 to 114	.....	3.38	4.17	.....	.....	.....
Lock-Valves								
Starved Rock	63	57	.....	.....	.....	.....	.....	.....
Marseilles	62	57	.....	.....	.....	61	56	.....
Lockport	.....	.....	.....	.....	.....	92	90	.....

Taintor Gates at Starved Rock.—The Taintor gates at the Starved Rock Dam were all completed and tested before the Waterway was turned over to the Government. The War Department Engineers made some tests of the flow through these gates partly open, varying from a small fraction of a foot to about 9 ft. With the water in the reservoir averaging about 4 to 5 in. below the level of the tops of the gates when closed, runs were made with the results shown in Table 4. Confidence can be felt in the reliability of the coefficients obtained.

TABLE 4.—COEFFICIENTS OF FLOW THROUGH TAINTOR GATES AT STARVED ROCK DAM

Item	Number of gates open	Height of opening, in feet	Coefficient
1	9	0.797	0.996
2	8	1.066	0.964
3	7	0.931	0.931
4	6	1.268	0.982
5	1	9.000	0.737

The openings under the gates were entirely submerged, except in the case of Item 5 where the lower edge of the gate was about 7.06 ft above the level of the lower pool.

There is a large pool up stream from the Starved Rock Lock, but the deep water is on the opposite side of the valley where it has just the depth necessary for navigation. Down stream, the water is carried away in the standard channel of the waterway 200 ft wide by 10 ft deep.

The consequence of this is that when the upper valves are opened to fill the lock, the up-stream water drops and does not regain the full head for about 5 min; therefore, for two-thirds of the time of filling, the flow is not under full head. Furthermore, when the lower valves are opened to empty

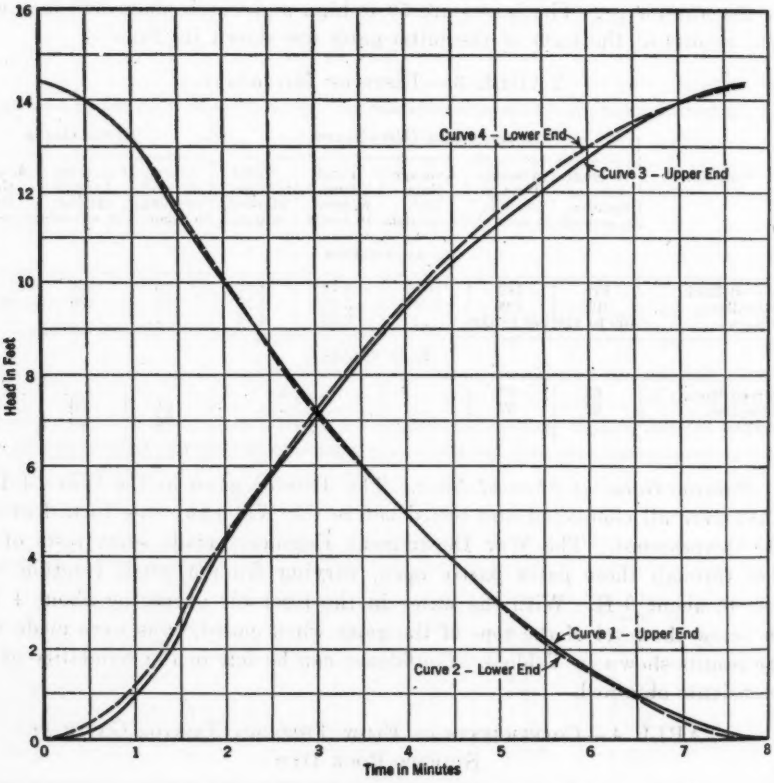

FIG. 22.—STARVED ROCK LOCK OPERATING TEST

the lock, the first rush of water lowers the down-stream flow for a few seconds. It then surges back and rises higher than normal, decreasing the head for emptying materially; this excess head down stream gradually lowers as the flow from the lock decreases.

In the past, trouble has been caused by surging of water in locks during filling or emptying, making it necessary to limit the rate of rise or fall in the lock. A careful study was made of this subject by L. D. Cornish, M. Am. Soc. C. E., and he came to the conclusion, that there should be more ports from the culverts into the up-stream half of the lock than into the down-stream half. He developed a formula for the spacing of the ports.

By referring to Fig. 2 in the paper, it will be seen that there are six ports from each culvert in the up-stream half of the lock, while there are only four in the down-stream half. It will also be noticed that these ports are not equal distances apart, there being a long distance near the center of the lock with no ports.

On account of this careful study, the tests were made to find not only the time of filling and emptying and the corresponding coefficients, but to see whether objectionable surging had been eliminated. The latter is much more important than a small difference in the coefficient of filling and emptying, as it generally takes considerable time to moor a vessel securely if the surging is great. In the tests, therefore, an attempt was made to determine the exact time of fall or rise of each foot at each end of the lock during each filling or emptying. An observer was stationed at each end of the lock with a tape line attached to a float. During the operation each observer would read the time on a stop-watch for each foot of rise or fall. The method was rather crude as the floats were not protected from the rough water in the lock during filling.

In Fig. 22 is shown some typical curves of flow for emptying and filling the lock. Curve 1 is for emptying the upper end and Curve 2 for emptying the lower end. Comparing these two curves, it will be seen that there is a slight surging, but not more than 1 in. from the normal at either end, because of a maximum difference in level of from $1\frac{1}{2}$ to $1\frac{3}{4}$ in. between the water at the up-stream and down-stream ends. The following flow coefficients were determined, from the time the valves were fully open, on the basis of difference in head:

	Emptying lock	Filling lock
Culverts, upper end, $C =$	0.76	0.77
Valves, upper end, $C =$	1.06	1.05
Culverts, lower end, $C =$	0.73	0.78
Valves, upper end, $C =$	1.03	1.06

It is seen that a coefficient of from 0.73 to 0.75 was obtained for the culverts and from 1.03 to 1.06 for the valves.

It was observed that at the beginning of the opening of the valves the water in the canal below the lower gates was first shoved away down stream and the level was lowered about 0.7 ft. It surged back quickly until it rose to 1.5 ft above the level of beginning of flow and then very slowly lowered so that for almost the entire time of emptying the lock the actual head causing the flow was materially less than the theoretical head used in the computations. Therefore, the actual coefficient should be considerably greater than the one shown.

Curves 3 and 4 in Fig. 22 are typical of flow for filling the lock, observation being made at both the up-stream and the down-stream ends of the lock the same as in the emptying test.

It will be noticed from these curves that the surface at the up-stream end is consistently from 2 or 3 in. above the surface at the lower end until just before the lock is full, when they practically coincide. It will be noticed that coefficients of 0.77 and 0.78 are obtained for the culverts and 1.05 and 1.06 for the valves. In this case, also, the actual head causing the flow is considerably less than the head used in the computations from the fact that upon opening the valves the head-water falls 0.8 ft in the canal above the gates before the valves are fully opened, and does not regain its level for 5 min; therefore, the lock is filling under a smaller head than that used in the computations.

When the valves and culverts were designed by the writer, it was hoped to obtain, as a possible maximum coefficient, 0.83 for culverts and 1.15 for the valves. It was realized, however, that seldom is a coefficient of more than 0.75 obtained for the culverts in a lock of this type, and as the valves did not reduce the flow in the culverts below this coefficient, it seems conclusive that they are working as well as larger valves, which answers Mr. Sargent's question about the operation of the valves.

A tow consisting of a large towboat, dredge barge, and derrick barge, coupled end to end and occupying about 300 ft, was locked through between two of the tests. To show how little surging there is during filling at full speed, this tow had out only one stern line and one bow line for the entire tow, and neither of these lines were taut at any time during the filling. Furthermore, the vessels did not move more than a foot away from the lock-wall during filling. The entire time from passing the lower gate going in to passing the upper gate going out was 20 min, and a part of this time was used in taking on water after the upper gate had been opened.

TABLE 5.—COMPARISON OF FLOW COEFFICIENTS THROUGH CULVERTS AND VALVES

Description	Pedro Miguel Lock, Panama Canal	Starved Rock Lock, Illinois Waterway
Flow Coefficients:		
Filling chamber with two culverts	0.65	0.775*
Emptying chamber with two culverts	0.67	0.745*
Filling chamber with side culvert only	0.82	.....
Emptying chamber with side culvert only	0.73	.....
Filling chamber with center wall only	0.52	.....
Emptying chamber with center wall only	0.66	.....
Filling lock-chamber with two culverts (for valves)	.....	1.055
Emptying lock-chamber with two culverts (for valves)	.....	1.045
Areas of Cross-section, in Square Feet:		
Valves	576	162
Culverts	510	226
Ports	1 285	350
Proportions:		
Valves	1.)	1.00
Culverts	0.88	1.40
Ports	2.23	2.16

*For culverts.

The coefficients obtained in filling and emptying the Pedro Miguel Lock[22] of the Panama Canal are compared with those of the Starved Rock Lock in Table 5, as are also the comparative areas of valves, culverts, and ports in the two structures. It will be noticed that the Panama locks have a culvert area 2.25 times, a port area 3.68 times, and a valve area 3.6 times as great as the locks of the Illinois Waterway. The Panama locks have a ratio of valve area to culvert area of 1 to 0.88, whereas the Illinois Waterway locks have a ratio of 1 to 1.4, and the Waterway has as high a coefficient of flow as the Panama Canal.

The spring sealing strips along the lower edge of the miter-gates to form contact with the miter-sill have given some trouble at the up-stream gates at Starved Rock Lock. When filling the lock a slight noise and vibration starts in the gates when the water in the lock has risen to within about 5 ft of the level of the water in the canal above the up-stream gate. This noise and vibration increases slightly for the next 2 ft of rise; then it gradually dies down and stops when the difference of level is somewhat less than 1 ft. Apparently, this vibration is caused by the pressure not being sufficient to keep the spring in close contact with the sill when there is a moderate difference in head. The same noise and vibration between these same levels occurs while emptying the lock. Some of the bolts attaching the spring seal to the gate have been broken by the vibration. It is singular that no noise or vibration occurs in the lower gates.

Bridges.—As stated in the paper, the State was required to replace twelve bridges across the Waterway. It was decided to have two of these bridges constructed by the State Division of Highways and the other ten by the Division of Waterways.

In Joliet, Ill., it was decided to construct four bridges of the rolling-lift bascule type. The fifth bridge in Joliet will be of the trunion bascule type similar to the Brandon Road Bridge. The Brandon Road Bridge was finished and thrown open to traffic in the latter part of 1932.

The cheapest type of bridge that could be used is the simple through-truss fixed bridge and whenever conditions would allow, this type was used. There are two bridges of this type. The fourth type used is a cantilever at Ottawa, Ill. This bridge, while costing slightly more than a simple through truss, was much better suited to this location on account of the long center span, as the Fox River enters the Illinois River immediately above the bridge. Furthermore, the elimination of falsework under the main span is a very desirable feature in the construction of the bridge.

Mr. Sargent has raised various points in discussion. The question as to the success of the future method of valve construction has already been answered. The valves operating under their own weight only, in closing, work with thorough satisfaction as the friction in the roller bearings is very slight.

A skin is used only on one side of the gates and is solely for economy. The reason for putting it on the down-stream side was to make it possible

[22] "Hydraulics of the Locks of Panama Canal," by R H. Whitehead, Vol. II, p. 200. of "The Panama Canal," by the late George W. Goethals, Hon. M. Am. Soc. C. E.

to put the miter-sill along the down-stream side of the gate instead of back beneath the gate.

The emergency dam at Lockport is so simple that there is little prospect of its giving trouble, and it has been operated and thoroughly adjusted. The recess under the up-stream leg is protected so that nothing can get in it but sediment, and this can be sluiced out at any time by connecting the recess with the down-stream pool at one end and the up-stream pool at the other. This will wash the chamber out thoroughly.

Lines of mooring posts have been provided at various levels in recesses in the side-walls of the locks so that lines can be changed as the barges rise or fall. As stated, all appreciable surging has apparently been eliminated in these locks so that very little mooring is needed.

Mr. Walker inquires about the success of the Venturi principle in the valves. He also inquires about the reasons for the adoption of the miter-gates with a rise of 1 to 2 in the sills and with parallel faces.

A very thorough study was made in the design of the gates for the Panama Canal Locks, and actual designs were made upon which prices were obtained. The result was that the type of gate adopted is the one that was shown to be most economical in that investigation. Inasmuch as the Illinois Waterway Locks are the same width as those at Panama, it is evident that the same condition obtains here. A complete account[23] of the investigations pertaining to the Panama Locks was published by Henry Goldmark, M. Am. Soc. C. E.

The detail of the miter-post is exactly the same as the quoin-post, except that the adjustable bearing pieces are alike in both miter-posts and that the convexity of the abutting surfaces are of a much longer radius.

Both Mr. Walker and Mr. Molitor criticize, severely, the method of analyzing the stresses in the side-walls in the part surrounding the culverts. Attention is called to the original paper under "Side-Wall Analysis," where the writer states, "it is not claimed that this method will give exact results, but it is believed to give results nearer the truth in a case of this kind than any other method." He is still of that opinion. Contrary to opinions expressed by Messrs. Walker and Molitor it is believed to be well worth the time spent upon it even if the results are approximations. Engineering structures are designed and built every day under assumptions just as uncertain as any made in this case.

[23] "Lock Gates, Chain Finders, and Lock Entrance Caissons," by Henry Goldmark, M. Am. C. E., in Vol. II, pp. 87–93, "The Panama Canal" by George W. Goethals, Hon. M. Am. Soc. C. E.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

DETERMINATION OF PRINCIPAL STRESSES IN BUTTRESSES AND GRAVITY DAMS

Discussion

BY W. H. HOLMES, ASSOC. M. AM. SOC. C. E

W. H. HOLMES,[48] ASSOC. M. AM. SOC. C. E. (by letter).[48a]—As stated in the "Introduction," the writer's study of the cause of cracks in the buttresses of a multiple-arch dam led to the development of a formula for determining the principal stresses at any point on any horizontal plane through the buttress. When inclined contraction joints were proposed for a high curved gravity dam, the formulas were expanded to include gravity dams arched in plan. These formulas greatly reduced the tedious work involved in the study of the stresses in the interior and along construction joints parallel with the axis of buttresses and gravity dams. The logical method of presenting the subject was first to show the development of certain formulas for the straight gravity dam along the lines proposed by the late Professor Cain, and then to derive the more general cases of gravity dams curved in plan and buttresses with tapering sections.

The discussions of the paper covered five main questions, or subjects, namely:

1.—The necessity for any analysis of stresses other than at the faces;
2.—The practicability of the formulas;
3.—The accuracy of any formula based on the trapezoidal law;
4.—Past use of Professor Cain's theories in practice; and,
5.—The selection of economic sections.

The personal opinion of several writers was expressed as to various details of design, but cases for which no general proof was given, will not be discussed.

NOTE.—This paper by W. H. Holmes, Assoc. M. Am. Soc. C. E., was published in January, 1932, *Proceedings*. Discussion on this paper has appeard in *Proceedings*, as follows: May, 1932, by Messrs. W. J. Stich, Hakan D. Birke, Dirk A. Dedel, Fred A. Noetzli, and Eugene Kalman; August, 1932, by I. M. Nelidov, Assoc. M. Am. Soc. C. E.; October, 1932, by Messrs. H. E. von Bergen and, Howard L. Cook; and December, 1932, by Messrs. Herman Schorer, A. Floris, W. C. Huntington, and Calvin V. Davis.

[48] Hydr. Engr., State Dept. of Public Works, Los Angeles, Calif.

[48a] Received by the Secretary November 25, 1932.

Professor Kalman has questioned whether any study of the stresses at some interior point was necessary. He then derives equations to prove for a triangular profile, and with reservoir filled, that there can be no maximum stress in the interior of the section. Mr. Birke also demonstrates that the vertical and horizontal normal stress and shearing stresses for a triangular section are a linear function of x. The writer's Equations (34), (35), and (36) prove that, for the triangular profile, the stresses are a maximum at the downstream face. Professor Kalman concludes his discussion of maximum stresses with the statement that if the profile is not triangular, there will be some stress distribution other than linear along straight-line sections, but that the maximum stress will probably appear at some point on the faces. However, he states that since dams are supposed to be continuous, homogeneous structures, he has never believed in searching the interior for extreme stresses.

The writer concurs with Professor Kalman that for continuous and homogeneous triangular profiles the stress will be a maximum at the faces and any further analysis based on the trapezoidal loading theory is useless, because the principal stresses at the faces can be computed directly.

Mr. Birke has derived certain formulas (Equations (87) to (107)) for determining principal stresses in a triangular section. These discussions only apply to a continuous triangular section, and have no application other than checking the writer's general formulas when applied to this particular case. It is erroneous to extend his deductions to an irregular section.

Many dams are not triangular, and a brief inspection of many of the larger gravity dams and buttresses would indicate that open cracks prevent the structures from being continuous; hence, some analysis should be made to determine the stresses in the irregular sections, and also to determine whether the stresses are a maximum at some point other than at the faces. Physical evidence indicates that there is some stress not found by analyzing stresses at the faces, that is causing buttresses to crack.

Professor Kalman questions the use of any analysis for adjustment of stresses in a section arched in plan, and has raised the old question of arch action in a gravity dam, curved in plan. He concludes that a designer who does not believe in arch action of a gravity dam will not design the dam as an arch, "a layout involving increased cost." Conversely, if a designer believes in some arch action, he should assume, according to Professor Kalman, that it compensates for "very moderate differences in the behavior of tapering blocks in arched dams and normal rectangular blocks in straight gravity dams," and therefore, should not consider "the tedious features of tapering blocks."

His arguments are not in accord with the facts, as may be illustrated by an example. The proposed San Gabriel Dam was to have been a gravity section, 492 ft high, arched in plan on a 1 400-ft radius, and was to have a crest length of 2 100 ft. The designer stated[49] that the primary reason for

[49] *Western Construction News*, July 10, 1927, p. 48.

arching the dam was that the cost was less, as better rock at less depth was found at that location.

A dam of this length cannot be considered as a monolith; hence, it will crack or open at the construction joints. It is logical to make such construction joints on radial lines. The distance between any two radial lines at the down-stream toe is 73% of the distance between the same lines at the up-stream face. This reduction of 27% of the base width at the toe could not be considered as a "very moderate difference." There is valuable property and a dense population below this proposed dam, and any reasonable mathematical study promoting safety should not be omitted because it is too "tedious."

Professor Kalman believes the writer's statement to be untrue that "a triangular profile with vertical up-stream face is safer than a rectangular profile having the same up-stream face and base." A triangular dam of such height that the maximum compression or shear equals the safe working strength of the material used, would not be safe if, later, the section was changed to a rectangular one having the same base.

For examples of triangular and rectangular dams as shown in Examples 1 and 2, the maximum shear in the rectangular profile is 39% greater than in the triangular profile. The first principal stress is also 39% greater in the rectangular profile than in the triangular profile.

Mr. Floris could not find any advantage in using an algebraic formula for deriving stresses in buttresses. Any one attempting to extend Professor Cain's numerical method, for other than the simplest cases, would find that a systematic grouping of certain factors as in the algebraic method would be a saving of time, and, what is of more importance, an assistance to greater accuracy.

The practicability of formulas using 1-ft increments was questioned by Messrs. Stich, Birke, and Dedel. Mr. Stich found it necessary by his method to use coefficients of eighteen significant figures for the calculations of stresses in the lower elevation of the San Gabriel Dam. The writer did not have the difficulty expressed by Mr. Stich in making the stress analysis of this dam. Results of these computations were given in Example 4, for the height of the dam equal to 100 ft. For a height of 492½ ft, no additional difficulty was found in confirming each of the checks, 1 to 4. Mr. Birke concurs with Mr. Stich that using planes 1 ft apart vertically is too cumbersome. These discussers appear to have overlooked the fact that if the moment in one plane is known to any desired degree of accuracy, it is a simple matter to determine the moment of a section 1 ft above or 1 ft below that section to the same degree of accuracy.

Both Messrs. Birke and Dedel criticized the methods developed, because these methods did not cover the one particular case in which they were interested, namely, buttresses having an enlarged up-stream face, as shown in Fig. 17. Mr. Birke found that the omission of these sections gave "results that are incorrect and not on the side of safety for structures having variable sections, such as buttress designs with flaring sections, haunches, pilasters, arches, etc."

If the rectangular base section for any dam were safe, it is impossible to conceive that by the addition of a small amount of material (haunch, etc.) near the up-stream face the structure would become unsafe. The writer is unable to follow Mr. Birke's arguments. Even his colleague, Mr. Dedel, concurs with the writer that the omission of the flanges produces a conservative design, instead of an unsafe design.

Mr. Dedel criticized the writer for proposing a "too conservative design" and, in Fig. 18, attempts to prove that his own designs are less conservative. The writer believes that Mr. Dedel's assumptions are less conservative, even to the point of being unsafe. It is time that a more conservative thought is given to the design of buttress dams, because the record of some of the larger buttress dams is not very satisfactory. Fredrik Vogt, Assoc. M. Am. Soc. C. E., has described the cracks in a number of multiple-arch dams,[50] and a repetition is unnecessary.

Both Mr. Birke and Mr. Dedel appear to have difficulty in analyzing a section having flanges or haunches. If they had examined a few of the many buttresses that have cracked they would have found the proper procedure to follow in determining what part of the base to include in the analysis. Many cases could be cited to show where open cracks exist in the arch ring and extend through the buttress near the up-stream toe of the dam, thereby eliminating the arch or haunch from any part of the analysis.

If the stress at the down-stream toe is called the first principal stress, continuing a fiber of stress to the up-stream face, the external force (water pressure) must be the first principal stress. If the buttress has an open crack at the foundation, the first principal stress at the up-stream edge of the block down stream from the crack is zero. If the diagram of the first principal stress was shown by Mr. Dedel for Fig. 18, it would have been approximately a rectangle. A crack near the heel would immediately reduce his rectangle of stresses to a triangle. Again, referring to Fig. 18, as far as can be determined from inspection, the diagram of the second principal stresses shows a straight-line variation. Such cannot be the case for even the simple triangular profile.

Mr. Dedel develops an equation for the stresses at a point below the haunch by simply ignoring the existence of the haunch and the full thickness of the buttress and haunch. His Equations (110) and (113), if applied to the face, are identical in value to the writer's Equations (26) and (27). Mr. Dedel's Equation (112) is identical to that in the paper, preceding Equation (26). It would seem that Mr. Dedel's entire criticism of the paper is that no formulas were developed to care for approximate solutions during preliminary stages of design, but when he attempts to make an exact analysis, he concurs that "the exact method of analysis similar to the author's method must be used." A curve to assist in the selection of a proper base width of approximate trial solutions to eliminate tension is described later in this discussion. Mr. Nelidov has derived analytical equations for buttresses having an abrupt change in section, by which Mr. Birke's and Mr. Dedel's particular case can be solved for a monolithical homogeneous section.

[50] "Shrinkage and Cracks in Concrete Dams," by Fredrik Vogt, 1930.

Mr. Stich has raised the question of checking Equations (26) and (27) and has stated that these equations are only true for special cases. It can be shown that S_u and S_d approach S_0; and also that p_u and p_d approach p_0 as Δ approaches zero. If the planes are taken as Δ distance apart, S_u and S_d approach equality and also p_u and p_d approach equality and, on Plane 0, are equal for all cases. Therefore, the equations as given are mathematically correct for all cases.

Mr. Cook takes exception to the basic assumption, namely, trapezoidal loading. This assumption is invariably made in the design of large dams, and will have to be used until something better, based on either a new theory or experimental data, is produced. He refers to a model test and states: "If the hypothesis of linear distribution of stress were true, the shearing stress would be distributed parabolically." Based on the trapezoidal law, the horizontal or vertical shear in a rectangular beam is distributed parabolically (the beam having both shear and moment at the section considered), but the maximum shear is an equation of a higher degree. An equation for maximum shear in an irregular section would be very complex. The shear is maximum on two planes, making 45° with the planes of principal stress, and is zero on the planes of principal stress. Shear deformation measurements, if not properly made, would give misleading results. The determination of maximum shear in a model by elastic deformations is not as simple as Mr. Cook infers. The photo-elastic method does show maximum shear, but as stated by Dr. Max Mark Frocht,[51] the solution of principal stresses is not simple. As Mr. Cook's shear hypothesis is in error, he cannot use it to prove that the trapezoidal law is in error.

The writer concurs with Mr. Cook that model tests may be an aid in the art of dam design, but thus far they cannot be substituted for analytical methods of determining stresses.

The writer cannot concur with Mr. Birke in the statement that all important dams recently built have been analyzed by the method for determining principal stresses, which are practical and theoretically correct.

The part of the buttresses of the Rodriquez Dam, built in 1930, do not have any inclined joints, and, in fact, designs of the same dam made in 1928 show only vertical normal stresses. It has been reported that the amount of reinforcing steel was increased after construction started on the Lake Pleasant Dam,[52] Sutherland Dam, and Stony Gorge Dam.[53] Furthermore, a paper on the details of design and construction[54] for the Stony Gorge Dam makes no mention of principal stresses.

Mr. Birke makes the statement in his discussion that if properly arranged contraction joints are placed along the trajectories of the first principal stress, and if the second principal stress is either compression or zero, there will be no tendency for the joints to open and, as the shear is zero, there will be

[51] *Transactions*, Am. Soc. Mech. Engrs., Vol. 53 (1931).
[52] *Western Construction News*, April 25, 1929, p. 199.
[53] *Loc. cit.*, August 10, 1928, p. 494.
[54] *Loc. cit.*, August 10, 1928, p. 490; and *Engineering News-Record*, July 11, 1929, p. 46.

no tendency to slide. Mr. Birke assumes, therefore, that the structure is able to take the stresses necessary for monolithic action and inquires why it should not be so designed. Shearing movement along actual cracks in the aforementioned dams, and in others,[55] refute Mr. Birke's theories in this respect.

Cracks in the buttresses of the Lake Hodges Dam have been observed by many engineers for years. Micrometer readings across the cracks have been made for ten years, and show a steady yearly increase in crack openings. Some of the cracks now (1933) are open about 0.3 in. It is obvious, therefore, that the buttresses of this structure are not acting as a monolith, since tension or shear cannot be transmitted across the open joints. This one case furnishes sufficient evidence to prove that a large buttress should be designed in separate units.

Various theories[56] have been advanced as to the probable cause of failure of materials, among them (1) maximum stress; (2) maximum strain; and (3) maximum shear. No theory yet advanced appears to account for all the physical phenomenon observed.

The maximum stress theory or a maximum working stress is commonly used in building codes, but it has been shown that most materials, if restrained laterally, will carry in compression many times the ultimate load for the same materials when unrestrained. Such observations would lead to the adoption of the maximum strain theory. If the maximum strain theory is used, Poisson's ratio must be included in all calculations.

After principal stresses, as defined in the paper, have been obtained by any convenient method, true stresses may be computed from the formulas given in most textbooks, namely, if T is true stress, f_1 and f_2 are principal stresses, and μ is Poisson's ratio:

$$T_1 = f_1 - \mu f_2 \quad \ldots\ldots (140)$$

and,

$$T_2 = f_2 - \mu f_1 \quad \ldots\ldots (141)$$

To eliminate tension near the up-stream face due to the deformation, f_2 be made equal to $\frac{f_1}{\mu}$, or,

$$f_2 = \frac{f_1}{\mu} \quad \ldots\ldots (142)$$

Using this criterion, the ratio of base width to height becomes large, and it probably would be a more economical design to use steel reinforcing instead of widening the section. Steel also should be used to care for stresses due to temperature changes and shrinkage, but this subject is beyond the scope of the paper.

Mr. Birke makes reference to the Big Dalton Dam as an example of a "well established" method of locating joints along the direction of principal stresses. The full amount of steel crosses the so-called joint in this dam,

[55] *Engineering News-Record*, January 31, 1924, p. 182.
[56] "Plasticity," Nadai, Engineering Society Monographs, p. 59.

and even one-third of the concrete is continuous. This so-called joint as shown in Fig. 23 is a misnomer, as it is only a line of weakness formed by inserting building paper from each face one-third the thickness of the buttress. Since this dam was built in 1929, and to date (1933), no appreciable quantity of water has been held in storage, no conclusions can be drawn as to the effectiveness of this untried joint in providing monolithic action without ill effects from the secondary stresses that are involved.

As a direct answer to Mr. Birke's question, an opening of a construction joint or a crack causes a redistribution of stresses; hence, no section larger than the distance between open joints or cracks should be analyzed as a monolith. The redistributed stress, in some cases, may be more than double the original stress.

The writer concurs with Mr. Stich and Mr. Birke that, in order to eliminate or decrease the effect of shrinkage, some joints parallel with the axis are necessary for massive concrete dams; but he cannot concur that a joint on the line of first principal stress is a "cure-all" for dam design. Hydrostatic pressure in the joint or shrinkage will alter the stress distribution, and the joint will no longer be a line of zero shear.

Mr. von Bergen has extended the formulas to indicate sections of minimum tension. A formula for sections having zero tension should be of value. Very simple tables or charts can be made to show the base width required

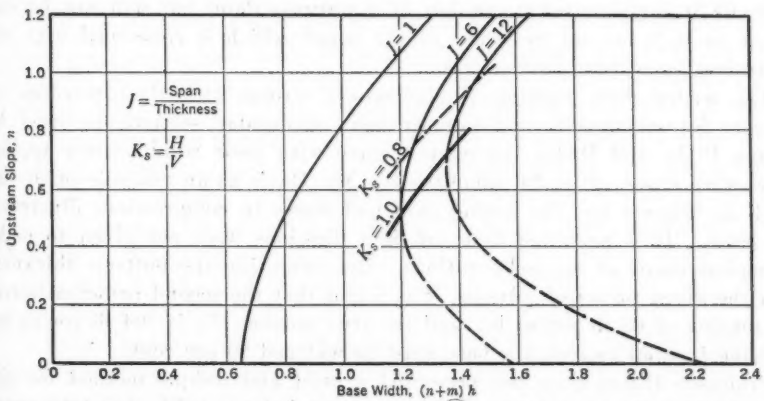

FIG. 31.—UP-STREAM SLOPE, BASE WIDTH, AND SLIDING FACTOR FOR STRAIGHT GRAVITY DAMS DESIGNED FOR ZERO STRESS NORMAL TO UP-STREAM FACE (NEGLECTING UPLIFT), WHEN FILLED WITH A LIQUID OF DENSITY OF J.

for a gravity section straight in plan when no tension exists in the heel, and when various weights of concrete and various assumptions of uplift are used.[57] If it is desired to design a buttress of continuous triangular profile that has no tension, approximate limiting conditions of base width may be tabulated.

The buttress type of dam has more variables than the straight gravity dam, but the curves shown in Fig. 31 were derived by making the following

[57] *Western Construction News*, October 25, 1928, p. 662.

assumptions: (1) The triangular section is a monolith; (2) the base is rectangular; (3) the weight of masonry is 150 lb per cu ft; (4) liquid in storage has variable densities; (5) no tension occurs in second principal stress at the up-stream face; and (6) Poisson's ratio is neglected. Assumption (4) eliminates the question of type of facing, but allows for the variable ratio of buttress spacing to buttress thickness.

An example will illustrate: Given $n = 0.5$; buttress specing, 60 ft; buttress thickness, 5 ft (or the equivalent of a liquid with a density of twelve times that of water acting on a gravity dam); $J = \frac{60}{5} = 12$. It is found that $m = 0.89h$. With an up-stream slope of 0.5 and a down-stream slope of 0.89, the base width is $1.39h$.

It will be observed that the economic limit of the up-stream slope for a density of 1 is 0; for a density of 6 is 0.5; and for a density of 12 is 0.75. The batter of the up-stream slope is limited more by the sliding factor than by any other condition.

If this example were designed on the basis that T_2 shall equal zero (Poisson's ration $= \frac{1}{6}$), it is found that $m = 1.46h$. Such a requirement would increase the base width by 40% and indicates that the use of reinforcing steel would make a more economical design.

Mr. von Bergen has discussed steel reinforcement. This subject is of economic importance in the design of a buttress dam, but will not be discussed, as it is beyond the scope of the paper, which is concerned with the determination of principal stresses.

The writer finds nothing fundamentally wrong with the procedure of analysis for continuous and homogeneous triangular sections as used by Messrs. Birke and Dedel, but cannot agree with their results when applied to practical cases. Fig. 30, submitted by Mr. Davis as an example of design which he believes has the second principal stress in compression, illustrates this point. It is regretted that sufficient dimsions were not given to make a complete check of his computations. By computing the buttress thickness from the given principal stresses, it is found that the second principal stress is a tension of 63 lb per sq in., and the true tension, T_2, is 104 lb per sq in. To make f_2 equal to zero, the base must be widened 20 per cent.

Professor Huntington has presented a brief and unique method for the direct determination of the vertical or horizontal shear and horizontal normal forces of a gravity section at a point. The writer's method first gives the equations for the vertical normal force, the shear distribution, and the horizontal force across the entire section. The stresses at any point are computed by substituting in these three equations. Professor Huntington's method is shorter if the stresses are desired only at a few points, but the writer's method involves less calculations if the stresses are desired at many points on the plane. A matter of more importance is that if either method is used safer dams will be designed.

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DISCUSSIONS

THE COMPENSATED ARCH DAM

Discussion

By EUGENE KALMAN, M. AM. SOC. C. E.

EUGENE KALMAN,[7] M. AM. SOC. C. E. (by letter).[7a]—Original ideas and interesting results that are distinctly thought-provoking are presented by the author. The paper can be subdivided into three parts, dealing with: (1) The compensation of the arch; (2) the design of compensated dams; and (3) some general aspects of arch dam design.

The study of isolated arches, free from bending moments and shears, embraces a formidable problem, which many consider impossible of solution. The conception of notches as a structural expedient is intriguing. The author shows that an arch, if conveniently shaped, can be compensated by inserting a system of properly proportioned notches, provided they will close perfectly under the action of the load, thus developing uniform compressive stresses on the face of the notches. What he does not show, and what is scarcely susceptible of proof, is that a series of blocks strung between two fixed abutments will become adjusted, under the action of the load, exactly in the manner expected by the author, and not in some of the other innumerable ways that are possible, at least according to logic.

Before proceeding to the discussion of the author's results, a few remarks should be made as to the methods used in the paper. By and large, the mathematical machinery, including the selection of the symbols, is cumbersome, even to those thoroughly trained in mathematics. In addition, the author is unusually sparing of hints as to his aims. Many readers of more than average preparation will have asked themselves; "What is he driving at?". Heavy, formulistic mathematical developments are liked by neither engineers nor mathematicians.

Using five pages, Mr. Karpov proves that an arch fixed at one end and free at the other, will have bending moments and shearing stresses unless t is

NOTE.—The paper by A. V. Karpov, M. Am. Soc. C. E., was published in April, 1932, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: October, 1932, by Messrs. R. A. Sutherland, Lars R. Jorgensen, L. T. Evans, and H. W Sibert.

[7] New York, N. Y.

[7a] Received by the Secretary August 22, 1932.

constant, and $T = p' R'$ is constant. This proof can be demonstrated much more simply. On the two faces of the arbitrary arch element in Fig. 13, moments and shears are excluded by assumption. Thus, the normal forces, T and $T + \frac{dT}{d\beta} d\beta$, as resultants of normal stresses uniformly distributed on the faces, will act at the centers of the faces. The force, $p' ds'$, acts in the center of the extrados element, ds', which may have an inclination, $\tan \omega$, with respect to the neutral axis and can be divided into components parallel

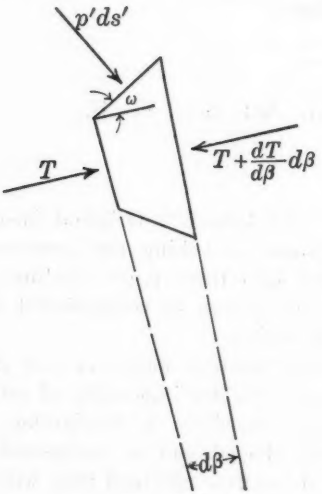

FIG. 13

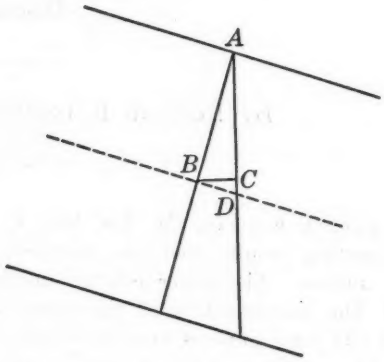

FIG. 14

and normal to the axis. The latter will meet the two lateral forces at their points of conjunction, so that this trio of forces will yield no bending moment with respect to that point, while the moment of the other component will be $p' ds' \sin \omega \frac{t}{2}$. This must be equal to zero. Hence, $\omega = 0$; that is, t is constant. Furthermore, the load acts in a direction normal to the neutral axis; and, evidently, $\frac{dT}{d\beta} = \frac{dT}{ds'}$. All this follows from the fact that $\omega = 0$. The condition of equilibrium of the three forces in the directions parallel and normal to the neutral axis yields $T + \frac{dT}{ds'} - T = \frac{dT}{ds'} = 0$; or T is a constant, and $T\, d\beta = p'\, ds' = p'\, R'\, d\beta$; or $p'\, R' = T$, respectively.

Here the assumption is made that there is no shear on the faces of any section, an assumption which is not stated explicitly in the author's proof. However, it is tacitly introduced in Fig. 4, in which the horizontal and vertical components of load and end reactions are apportioned to cover the normal stress, T, only.

In Mr. Karpov's exposition no use is made of his assumption, $M = M_1 + M_2 = 0$, as expressed by Equation (5); only the assumption, $\frac{dM}{ds'} = 0$, as expressed by Equation (16), is utilized, which means that in an arch fixed at one end and free at the other, the static condition, in which, t and T are constants, and $T = p' R'$, admits the presence of an invariable arbitrary moment, **M**, along the entire arch. This is also seen in the writer's proof, since the relation,

$$p' ds' \sin \omega \frac{t}{2} + \mathbf{M} - \mathbf{M} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (109)$$

does not alter the results. Equation (109) expresses the absence of resultant bending moments if there are two moments, **M**, that are equal and of opposite signs, acting on the two faces of the element. The presence of invariable moments is excluded only in a later stage of the investigation, when the notched arch span between two fixed abutments is analyzed.

The length of the author's demonstration as compared to the foregoing short derivation is due to the fact that instead of treating an arch element, he deals unnecessarily with the entire portion of the arch between an arbitrary section and the free end. Substantial "short-cuts" are possible even along that line of thought. For instance, that t is a constant can be inferred immediately from Equation (19), if the following two relations are considered:

$$\frac{d\left(\frac{t_1}{2}\right)^2}{ds_1} = \frac{d}{ds_1}\left[(x_1 - x_1')^2 + (y_1 - y_1')^2\right] = 2(x_1 - x_1')\left(\frac{dx_1}{ds_1} - \frac{dx_1'}{ds_1}\right) + 2(y_1 - y_1')\left(\frac{dy_1}{ds_1} - \frac{dy_1'}{ds_1}\right)$$

and,

$$(x_1 - s_1')\frac{dx_1}{ds_1} + (y_1 - y_1')\frac{dy_1}{ds_1} = 0 \quad \ldots\ldots\ldots\ldots (110)$$

Equation (110) follows directly from Equations (1) and (2), or from elementary geometry in Fig. 14, which shows that,

$$(x_1 - x_1')\frac{dx_1}{ds_1} = \overline{BC}\,\frac{\overline{AC}}{\overline{AB}}\,; \text{ and } (y_1 - y_1')\frac{dy_1}{ds_1} = -\overline{AC}\,\frac{\overline{BC}}{\overline{AB}}$$

Similarly, the exposition of the theory of elasticity in Section IV is too mathematical and unnecessarily long. An arch, fixed at one end and free at the other, is of such a shape that, under the action of a well-defined load, only a uniformly distributed compression of constant magnitude develops on any of its sections. In such a case, however, it is self-evident that the individual displacement of one face of a given elementary block relative to the other face consists merely in a translation, without any rotation. Consequently, the deflection of the free—say, right—end section due to the

deformation of the entire arch will also be pure translation. Its magnitude normal and parallel to the chord of the arch (depending on the individual displacement of the right face of an elementary block relative to its left face), is $\frac{dy}{ds}\frac{T}{Et}\,ds = \frac{T}{Et}\,dy$, and $\frac{dx}{ds}\frac{T}{Et}\,ds = \frac{T}{Et}\,dx$, respectively, the chord being the X-axis. Therefore, the total displacement of the free end section in the two directions, due to the total deformation of the arch, will be $\frac{T}{Et}\int dy$ and $\frac{T}{Et}\int dx$, or 0, and $\frac{TL}{Et}$, respectively, in which, L denotes the length of the chord. Furthermore, as is known universally, temperature changes and shrinkage have analogous effects on the deflection of the arch; therefore, the information in the latter part of Section IV, including Equations (47) to (52), is superfluous. Furthermore, it is certain to confuse many readers because of the unnecessary digression involved in the mention of a moment generated by temperature changes in the elastic center of the arch, a fact that has no bearing at all on the subsequent developments in the paper. Substitution of that part through a single statement to the effect that the total translation of the free end section in the direction of the chord equals $2\alpha_m\left(\frac{T}{Et} + c_T\,T_T\right) + (D_{haL} - D_{haR})$, would add much to the clarity of this part of the paper.

The decrease in the opening of the notches, due to uniform compression and temperature changes, is evaluated in Fig. 7 and the supporting text. The writer doubts the necessity of "proving" Equation (57) by applying to it such complex mathematics. Underlying the author's reasoning, evidently, is the tacit assumption that, owing to the infinitesimal magnitude of the angle, $d\alpha$, the lines, $C\,A$ and $D\,B$ (Fig. 7), are straight lines. In fact, if they were circular segments, the line, $A\,B$, would become a curved line corresponding to $A'\,B'$ in Fig. 7, under the action of uniform compression. This would affect, seriously, the principal assumption of the paper, namely, that the joints of the closed notches will be straight lines as would be the case if uniform normal stresses and absence of shearing stresses are assumed. On the other hand, if the lines, $C\,A$ and $D\,B$, are treated as straight lines, Equation (57) follows immediately from the similarity of the diagrams.

Likewise, the method of deriving Equations (67) and (68) is unnecessarily cumbersome. These formulas are quite interesting and, from a theoretical point of view, valuable, but, unfortunately, few engineers will be tempted to fight their way through the skillful but wearisome deductions. Hence, the following graphical and simple derivation is presented.

Let the arch be considered fixed at the right end, as shown in Fig. 6(*b*), and free at the left end. First, consider the deflections of kinematic character; that is, the deflections due exclusively to the closing of the notches.

In Fig. 15, Notches 1, 2, etc., close, one after another; the parts of the arch to the left of the hinges, 1, 2, etc., will undergo successive translations parallel to the left faces of the notches, 2, 3, etc. In this process of successive translations, Point i' of the extrados describes a certain path which is represented schematically in Fig. 16, by the broken line, $O\,I'$, composed of small chords,

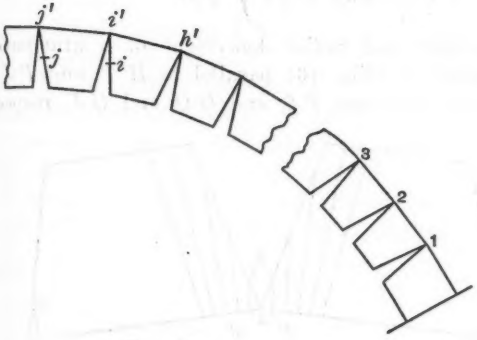

FIG. 15

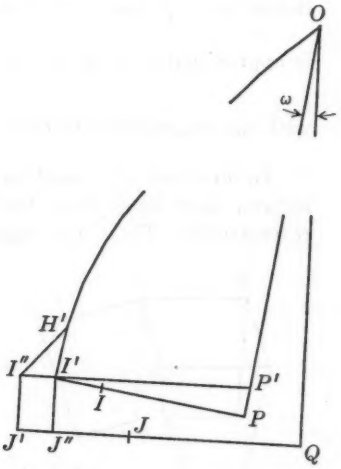

FIG. 16

the last of them being derived by rotation around the center, h', in Fig. 15. Represented in Fig. 16 by $H'\,I'$, it is of the magnitude, $ds' \int_1^h \phi\, ds$, and has the direction normal to that of the chord, $h'\,i'$ (Fig. 15) immediately prior to the rotation around h'. The path of the hinge, j', may be represented by the same broken line as far as H', and, from that point, it will continue to I'' ($H'\,I''$ being parallel to the left face of the notch, i'), and then to the point, J', the chord, $I''\,J'$, being of the magnitude, $ds' \int_1^i \phi\, ds$, while its direction is normal to that of the chord, $i'\,j'$, immediately prior to the rotation around i'. During the rotation of the block, $h'\,i'$, around the hinge, h', the chord, $i'\,j'$, does not change its direction; thus, the chords, $H'\,I'$ and $I''\,J'$, have the directions of the radii of curvature of the arch in the points, i' and j', a fact which will be utilized later in this discussion.

The deflection at the point, i, of the neutral axis, can be represented by the same broken line, $O\,I'$, completed by a segment, $I'\,I$, representing the tangential displacement of Point i, due to the rotation of the block, $i'\,h'$, around h'. The direction of $I'\,I$ is normal to $H'\,I'$; its magnitude is $\frac{1}{2}\,t \int_1^h \phi\, ds$. By analogy, the deflection of the arch in the point, j, may be represented by the broken line, $O\,J'\,J$, the segment, $J'\,J$ being normal to $I''\,J'$,

while its magnitude is $\frac{1}{2}\, t \int_1^i \phi\, ds$. It is easy to see that the magnitude of $I'' I'$ is negligible when compared to both $I' I$ and $I' H'$, so that the broken line, $H' I'' J'$ can be substituted by $H' I' J''$, the segment, $I' J''$, being parallel to $I'' J I$. This is seen from Fig. 17, which indicates that the tangential deflection of j' due to the rotation of i' around h' (that is, the segment, $I'' I'$), is approximately $\frac{1}{2}\, \phi_i\, ds'_i \times H' I'$, or $\frac{1}{2}\, \phi_i\, ds'_i \times ds'_i \times \int_1^h \phi\, ds$, as compared with the magnitude of $H' I'$, or of $I' I$, which is $\frac{1}{2}\, t \int_1^h \phi\, ds$.

To evaluate the total tangential and radial deflections of a kinematic nature, draw lines from the point, O (Fig. 16) parallel to $H' I'$ and $I' J''$, respectively. Then, the segments, $O P$ and $P I$, and $O Q$ and $Q J$, respec-

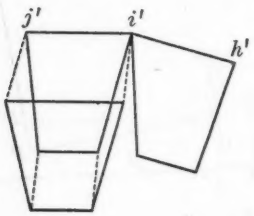

FIG. 17

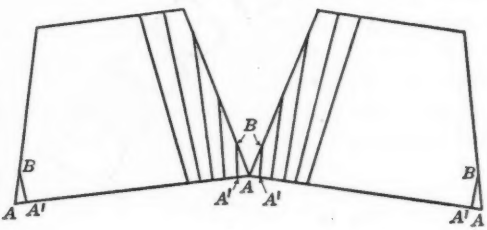

FIG. 18

tively, will be the radial and tangential deflections at the points, i and j, respectively. Next, drawing a line parallel to $J'' Q$ through I', the difference of the deflections relating to Points i and j, will be:

$$\Delta_T = Q J - P I, \text{ and } \Delta_R = I' J'' - P' P$$

Developing these relations further, the following results are obtained:

$$\Delta_T = (Q J'' - P I') - (J J'' - I I') = D_R\, \omega - (\tfrac{1}{2}\, t \int_1^i \phi\, ds) - (\tfrac{1}{2}\, t \int_1^h \phi\, ds)$$

$$= \frac{D_R}{R}\, ds - \tfrac{1}{2}\, t \int_h^i \phi\, ds = \frac{D_R}{R}\, ds - \tfrac{1}{2}\, t\, \phi\, ds$$

and,

$$\Delta_R = I' J'' - P I' \omega = ds' \times \int_1^i \phi\, ds - (D_T + \tfrac{1}{2}\, t \int_1^h \phi\, ds)\omega$$

$$= ds' \int_1^i \phi\, ds - (D_T + \tfrac{1}{2}\, t \int_1^i \phi\, ds)\, \frac{ds}{R}$$

Since the magnitude of the angle, $P I' P$, is equal to that of $P O Q$, which is equal to the angle, ω, between the radii of curvatures in the points, i' and j', of the extrados (as explained in the foregoing), the derivatives of the tangential and radial deflections will be,

$$\frac{dD_T}{ds} = \frac{\Delta_T}{ds} = \frac{D_R}{R} - \tfrac{1}{2}\, t\, \phi \quad \ldots\ldots\ldots\ldots\ldots\ldots (111)$$

and,

$$\frac{dD_R}{ds} = \frac{\Delta_R}{ds} = -\frac{D_T}{R} + \int_1^s \phi\, ds \quad \ldots\ldots\ldots\ldots\ldots\ldots (112)$$

due to the relation,

$$ds' - \tfrac{1}{2}\, t\, \frac{ds}{R} = ds\, \frac{R'}{R} - \tfrac{1}{2}\, t\, \frac{ds}{R} = ds$$

These formulas account only for the displacements implied in the kinematic closing of the notches; all stresses are excluded from the arch at this stage of the investigation. Then, as a consecutive step, the deflections due to the effect of the uniform thrust, may be determined as follows: Suppose that each block is acted upon, successively, by the invariable thrust, immediately after the notch at the right of that block has been closed. This would mean a scheme of deflections due alternately to kinematic and mechanical processes. Then, for Points i and j, a broken line will result, which is somewhat different from that represented in Fig. 16. The ultimate position of i will be a certain point, I''', on the corresponding line, $I\,P$, at a distance, $ds_i\left(\frac{T}{Et} + c_T\, T_T\right)$, from I.

The compression of the block, $h'\,i'$, involves, of course, the immediate displacement of j in the same direction and in the same amount as $I\,I'''$, on the corresponding line, $J\,Q$. Subsequently, the notch, i', will close, a process which does not bring about any change in the relative positions as determined in studying the kinematic deflections. As a last step, Block $i'\,j'$ will be subjected to the action of the thrust, such action involving an additional displacement of j' along the line, $J\,Q$, of the amount, $ds_j\left(\frac{T}{Et} + c_T T_T\right)$. It is seen, therefore, that the thrust does not effect any change in Equation (112), for the derivative of the radial deflection, whereas the derivative of the tangential deflection (Equation (111)), will be decreased by the difference of the tangential displacements of the points, i and j, due to the thrust, which corresponds to the difference in the lengths of the axis. The proportion of these differences being $ds\left(\frac{T}{Et} + c_T\, T_T\right)$ to ds; that is, $\frac{T}{Et} + c_T\, T_T$. Equations (67) and (68) thus appear to be proved, after the obvious substitution of β for ϕ.

These remarks are merely intended to show that cumbersome, abstract, mathematical deductions can easily be replaced by shorter methods that are more attractive, because each step in the reasoning has a definite, clear bearing on the engineering problem treated; thus, they do not imply any criticism of the results.

The paper contains one error, however, and a few inconsistencies, to which attention must be called.

In order to obtain the solution of Equations (67) and (68)—that is, an expression for β in the function of the radial or tangential deflections, or *vice versa*—the author derived, and subsequently solved, Equation (71). This step is not warranted as it involves an erroneous solution of Equations (67) and (68). That system of simultaneous differential equations is quite at variance with the indeterminate differential equation, (71), which contains the derivatives of two independent unknown functions. The former system has a well-defined set, or pair, of solutions if β is the independent variable. Equation (71), on the contrary, admits of an infinite number of solutions. In fact, if any arbitrary function is selected for one of the two unknowns, D_T *or* D_R, Equation (71) admits of a definite solution for the other. Among all the erratic and absurd couples of functions that may be obtained in this manner for D_R and D_T, there will be only one couple satisfying Equations (67) and (68). From this, one may infer that it will not be easy, if at all possible, to produce this one pair of functions by considering Equation (71), instead of treating the total system, Equations (67) and (68). As a matter of fact, and most unfortunately, Equations (72) and (73) claimed by the author to be the correct pair, are actually not the correct ones. They do satisfy Equation (71), but they do not satisfy the system of Equations (67) and (68). In fact, they do not satisfy either of those two differential equations, as a simple substitution shows. They cannot satisfy either of them because this would imply that their right members, and thus, also, their left members, should be equal to zero, which consequently implies that both D_T and D_R must be constants.

The correct pair of functions can be obtained by reducing the system of two unknowns to one differential equation with one unknown. For instance, by differentiation of Equation (67), the following relation will result:

$$\frac{d^2 D_R}{ds^2} = -\frac{1}{R}\frac{dD_T}{ds} + \frac{D_T}{R^2}\frac{dR}{ds} + \beta \quad \ldots\ldots\ldots\ldots (113)$$

Elimination of D_T and its derivatives from Equations (67), (68), and (113), will yield the differential equation:

$$\frac{d^2 D_R}{ds^2} + \frac{R'}{R}\frac{dD_R}{ds} + \frac{1}{R^2} D_R = \frac{1}{R}\left(\tfrac{1}{2} t\,\beta + R\,\beta + R' \int^s \beta\, ds + \frac{T}{E\,t} + c_T\, T_T\right) \ldots (114)$$

From Equation (114), D_R may be determined, at least theoretically, if β is known. Naturally, it is hopeless to attempt the solution, because of the complexity of Equation (114), and for the reason that β, and especially R, will not be given as explicit functions of s. A graphical solution can be obtained, although the procedure will still be very wearisome, in fact, almost beyond human endurance. Conditions are simpler if D_R is given and β is to be determined, in which case Equation (114) will be used in the form:

$$\left(1 + \frac{t}{R}\right)\beta + \frac{dR}{ds}\int^s \beta\, ds = \frac{d^2 D_R}{ds^2} + \frac{1}{R}\frac{dR}{ds}\frac{dD_R}{ds} + \frac{1}{R^2} D_R - \frac{1}{R}\left(\frac{T}{E\,t} + c_T\, T_T\right) \ldots\ldots\ldots\ldots (115)$$

This is somewhat easier to treat, since it is of the first order in $\int^{S} \beta\, ds$, although analytically it cannot be solved, as R and D_R will not be known in explicit form.

Another inconsistency occurs in the derivation and enunciation of Equation (69), that is, $\int_0^S \beta\, ds = 0$, deduced from Equations (67) and (68), and claimed by the author to hold for symmetrical arches. As shown previously, Equations (67) and (68) hold for any kind of arches fixed at one end and free at the other end, either symmetrical or unsymmetrical, no matter what arbitrary assumptions are made as to position and magnitude of the notches. Hence, for such an arch, the algebraic sum of the notches need not, necessarily, be equal to zero; in other words, Equation (69) need not hold. If, however, the arch is to be compensated—that is, if the notches are to be arranged so that their closing will not result in a rotation of the free end section—Equation (69) holds true for that arch no matter whether it is symmetrical or unsymmetrical. Then it need not, and should not, be derived from Equations (67) and (68). It follows simply from the very definition of the compensated arch; in fact, it is the definition of the compensated arch.

If the conception of the compensated arch means that the free end section does not undergo any rotation, then Equation (69) must hold, since it is the expression for the amount of the rotation of the free end. If, furthermore, the term, "compensated arch" refers to an arch in which the kinematic closing of the notches on one side, and the thrust on the other side, cause the free end section to translate in the direction of the chord by equal amounts and in opposite senses, then it is obvious, that, in addition to Equation (69), the following relations must hold:

$$\int^{S} \beta\, Y\, ds + 2\, \alpha_m \left(\frac{T}{E\, t} + c_T\, T_T \right) + (D_{haL} - D_{haR}) = 0 \quad \ldots . (116)$$

and

$$\int_0^S \beta\, Z\, ds = 0 \quad \ldots . (117)$$

the latter referring to unsymmetrical arches. In Equations (116) and (117) Y and Z represent the distances of the notches—situated partly on the extrados and partly on the intrados—from two axes, that of Y being the chord of the arch. The foregoing additional criteria of the compensation are not mentioned in the paper. This is unfortunate, because they play the most important part in judging the possibilities of the method under discussion.

Still another point of the paper should be discussed, namely, the derivation of Equation (68) through equalizing the external and internal work in the arch. The author's reasoning holds, under certain conditions, for a part of the arch extending from the fixed end to an arbitrary section characterized by the arch length, s. Thus, Equation (70) holds if the upper limit of the integrals is changed from S to s, and then Equation (68) follows

from Equation (70). This is not true for Equation (70) with S as the upper limit. The equality of the definite integrals of two functions between two given limits does not imply the identity of the functions. Another indication of the weakness of Equation (70) as it stands, is the presence of the definite integral, $\frac{1}{2}\,t\int_0^S \beta\,ds$, which, according to Equation (69), is equal to zero, and could be dropped without influencing the validity of Equation (70). In this case, Equation (68) would not evolve, even if the equality of the two definite integrals on both sides mean the indentity of the functions included under the integration signs.

There are other grounds for questioning the validity of Equation (70), which call for clarification of the manner in which the arch deforms. It must be borne in mind that a given load can cause the compensated arch between two fixed abutments to reach its final deformation in two different ways. One is an idealized procedure consisting of the following successive steps: (1) Temporary removal of the left abutment; (2) kinematic closing of the notches, without applying the load; (3) application of the given load on the extrados, and contemporary application of the thrust, T, on the free left-end section; and (4) since the arch is compensated, the effect of Steps (2) and (3) will be to keep the left-end section in its original position; hence, in Step (4), it is possible to replace the left-end abutment in its original position, in touch with the left-end section.

In contrast with this idealized process (which is well-defined, clear, handy for a quantitative analysis, and is used in the paper), the actual manner in which an arch deforms and is adjusted, between two fixed abutments, is unknown, and thus not susceptible even to a quantitative treatment. In the idealized process, the increase and the decrease in the chord length, affected by the closing of the notches and by the action of the load, respectively, have been allowed to take place in succession; but, actually, the gradual increase and decrease in the length of the chord must balance themselves at all times. This may be visualized as follows: At first, the effective arch will consist merely of a catenary passing through the connecting corners of the blocks. A certain initial compression will develop along this catenary, such that the original lengths, A-A, would become shortened to A'-A' (Fig. 18), and a centripetal displacement of the blocks would occur as they tended to fill in the space, B-B. Thus, the straight lines of contact, $A'B$, will become successively more and more inclined, as reproduced schematically in Fig. 18. Compression along the successively developing catenaries involves progressive decrease in the length of the chord; and adjustment of the blocks along contact lines of increasing inclination involves progressive decrease of curvature; that is, increase in the length of the chord. This increase and decrease must balance at all times during the process of deformation and adjustment.

The foregoing conception is necessarily a kind of fiction. No one knows whether it represents a picture even approximately true. Possibly, the actual process is not continuous, but discontinuous, involving occasional slips or abrupt changes in positions or stresses; possibly, the actual process occurs

in different ways when it is repeated, by alternately releasing and loading the arch; possibly, the actual process is not even reversible (the gradual removal of the load may not imply the exact reversal of the process of adjustment during the gradual loading, either as to stresses, displacements, or time); possibly, the arch will not even return to its original position when released from the load, because the blocks may hinder each other in their effort to return to their original strainless condition; and, it is possible, above all, that under the action of the load, a final adjustment will take place which is quite different from the uniform distribution of compressive stress on the sections, as assumed in the paper. Everything may happen in a structure composed of loose blocks. Experimental studies would be very instructive from several points of view. The successive development of the compression surfaces may not be susceptible of optical measurements; but the strains could be estimated from the stresses, if a series of observations by the photo-elastic method would disclose the stress distributions in the successive phases of adjustment.

Under such circumstances, it seems to be unwarranted to evaluate the amount of work by using the formulas in this paper. The expression, $\frac{1}{2} P \delta$, for the work, is derived on the assumption that while the force increases uniformly from 0 to its end value, P, the displacement of its point of application in the direction of the force also increases uniformly from 0 to its end value, δ. This assumption is warranted for ordinary, continuous structures; but the principle of superposition involved is not warranted in the case of a structure made up of isolated units. The author does not, and cannot, present any facts to prove that the deflections increase uniformly with a uniform increase of the loads and stresses.

After having pointed out the possible errors, the next question concerns the practical aspects of the problems involved. For example, it will be necessary to form an idea as to the dimensions of the notches. In the following discussion, this magnitude will be evaluated, assuming that the arch is semicircular and that all notches are equi-distant and of equal magnitude. Such assumptions will yield characteristic results without computations that are too intricate.

The assumption of equi-distant notches of the same magnitude, ϕ, satisfies Equation (69). Calling ω the magnitude of the central angle which defines the distance of the notches, Equation (113) requires that:

$$\phi\left(R-\frac{t}{2}\right)\left\{\sin(n+1)\,\omega+\sin(n+2)\,\omega+\ldots+\sin 2n\,\omega\right\}$$
$$-\phi\left(R+\frac{t}{2}\right)\left\{\sin\omega+\sin 2\,\omega+\ldots+\sin n\,\omega\right\}=\tfrac{1}{2}\,\delta\quad\ldots.(118)$$

in which, δ is the rib-shortening of the arch, and $4n$, the number of notches in the half circle. Equation (118) may also be written:

$$\phi\left(R-\frac{t}{2}\right)\sum_{i=1}^{n}\cos i\,\omega-\phi\left(R+\frac{t}{2}\right)\sum_{i=1}^{n}\sin i\,\omega=\tfrac{1}{2}\,\delta\quad\ldots.(119)$$

From mathematics,

$$\sum \cos i\,\omega = \frac{\sin \frac{n\,\omega}{2}}{\sin \frac{\omega}{2}} \cos \frac{n+1}{2}\,\omega$$

and,

$$\sum \sin i\,\omega = \frac{\sin \frac{n\,\omega}{2}}{\sin \frac{\omega}{2}} \sin \frac{n+1}{2}\,\omega$$

If the number of the notches increases, as assumed in the paper, the values of $\frac{1}{2}\,n\,\omega$ and $\frac{1}{2}\,(n+1)\,\omega$ approach 22° 30′, and for all practical purposes:

$$\frac{\sqrt{2}}{4}\,\phi\left(R-\frac{t}{2}\right)-\frac{2-\sqrt{2}}{4}\,\phi\left(R+\frac{t}{2}\right)=\tfrac{1}{2}\,\delta\sin\tfrac{1}{2}\,\omega$$

or,

$$\phi\,(0.33\ldots R-0.5\,t)=\delta\sin\tfrac{1}{2}\,\omega$$

Substituting angles for the small sines, the proportion of ϕ and ω will be given approximately by the formula:

$$\frac{\phi}{\omega}=\frac{\delta}{\frac{2}{3}\,R-t} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (120)$$

To apply this result to a practical case, let $R = 300$ ft; $t = 15$ ft; and $p = 15$ lb per sq in. Then, $\sigma = \frac{307.5}{15}\,15 = 307.5$ lb per sq in. The rib-shortening due to the load alone is:

$$\delta = 2\,R\,\frac{\sigma}{E} = 600\ \text{ft}\ \frac{307.5}{2\,000\,000} = 0.0922\ \text{ft}$$

Hence,

$$\frac{\phi}{\omega}=\frac{0.0922}{\frac{2}{3}\,300\ \text{ft}-15\,\text{ft}}=0.0005$$

Assume, for instance, block lengths of 20 ft. The opening of the notches will then be:

$$\phi = 0.0005\,\frac{20\ \text{ft}}{300\ \text{ft}}=\frac{1}{30\,000}$$

implying an angle of about 7 sec of arc. Since the thickness of the arch is 15 ft, the foregoing value means that in the V-shaped void of a notch, having a height of 15 ft, the base will have a length of only $\frac{1}{160}$ in.

Such magnitudes, of course, are not practicable. It is not possible to cast two concrete walls with plane, vertical surfaces so that at 15 ft from their common origin, their distance should be $\frac{1}{160}$ in.

It is conceded that: (*a*) Equation (120), derived on the assumption of ever-decreasing block lengths, may require some revision, if block lengths of 20 ft are used; (*b*) the arches may be far from being circular; and (*c*) their

dimensional characteristics may be different from those assumed in the foregoing computations. Consequently, the numerical results obtained herein are susceptible of changes; but their magnitude will not change substantially.

Next reverse the line of thought heretofore followed and investigate the consequences of assuming notches of reasonably large dimensions and of dropping, at the same time, the postulatum of equal and equi-distant notches throughout the arch.

The first thing to do is to establish a minimum practicable opening for the notches. Such a minimum, of course, is subject to differences of opinion, but it will be difficult if not impossible, to form cast-concrete plane surfaces, with angles smaller than about $2\frac{1}{2}$ degrees. Hence, this arbitrary or empirical quantity will be accepted tentatively. In the foregoing example of the circular arch, such an angle implies a base length of about 4 in. in the V-shaped voids.

In order to study the arrangement of the notches further, Equation (115) may be written in the following form:

$$\Sigma \phi \times Y_1 - \Sigma \phi \times Y_2 = \tfrac{1}{2} \times 0.09 \text{ ft} \quad \ldots\ldots\ldots\ldots\ldots (121)$$

in which, the quantities, Y_1 and Y_2, are the distances, from the chord, of the hinges of the notches, situated on the down-stream and up-stream faces, respectively, as shown in Fig. 19. Each sum in Equation (121) may be

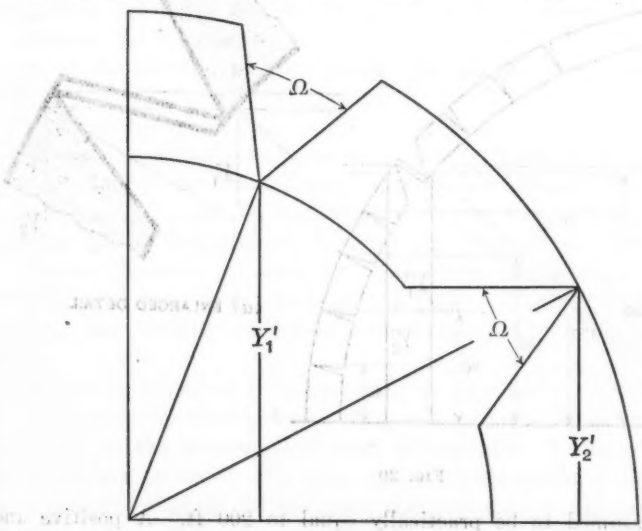

FIG. 19

replaced by a single product, composed of the concentration, or sum of the angles of the relative notches and an appropriate distance which may be termed their "middle distance." The summations of the up-stream and down-stream notches are equal, of course. If these sums are denoted by Ω, Equation (121) may be written:

$$Y'_1 - I'_2 = \tfrac{1}{2} \times 0.09 : \Omega \quad \ldots\ldots\ldots\ldots\ldots\ldots (122)$$

It is assumed that no notch should be smaller than $2\frac{1}{2}°$; therefore, the sums, Ω, will always be greater than $2.5 \times \frac{\pi}{360}$, and the difference of the two up-stream and down-stream "middle" co-ordinates, Y'_1 and Y'_2, will be always smaller than about 2 ft, if there is only one positive and one negative notch in the half arch; with the number of notches increasing, this difference decreases rapidly. In all cases, openings indicate that the two "middle" co-ordinates are practically of equal lengths. In other words, the radial vectors corresponding to the "concentrated" positive and negative notches are rather close to each other. Since at least one positive and one negative notch must be between the two radial vectors, and since these notches must balance all the other positive or negative notches outside the radii, it is clear that they will be quite large and will take almost all the space available between the radii.

This may be seen from Fig. 20, in which the arch is the same as the one treated previously. It will be assumed that notches with $2\frac{1}{2}°$ openings are situated at the distances, 45, 90, 135 and 170 ft from the chord, while Y'_1

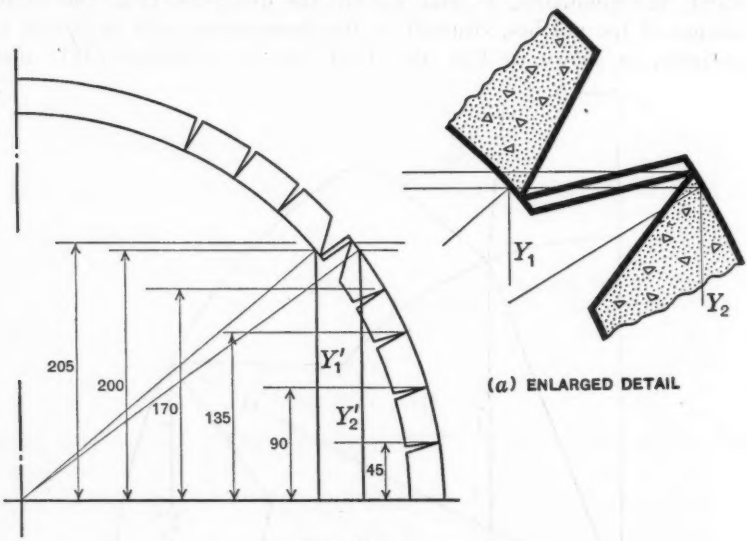

FIG. 20

and Y'_2 are assumed to be practically equal to 200 ft. A positive and a negative notch must be placed between the radial vectors corresponding to Y'_1 and Y'_2. Thus, the greatest distance of that negative notch from the chord cannot be more than about 205 ft. Noting the magnitude of that notch as x, and recalling the definition that the "middle distance" (that is, Y'_2), multiplied by the sum total of the negative notches is equal to the sum of the products of notches, by their distances from the chord, the following will result: $2\frac{1}{2}(45 + 90 + 135 + 170) + 205\,x = (4 \times 2\frac{1}{2} + x)\,200$; which gives $x = 72$ degrees.

Because of the fact that the sum of the positive notches must be equal to that of the negative notches, without regard to their distribution, it follows that the positive notch situated between the two radial vectors and balancing the other positive notches with respect to Y'_1, will be another angle of substantial magnitude. Thus, if it is at all possible to place both these wide notches between the two radial vectors, it is clear that there will be only a narrow transverse beam between the two notches, entirely isolated from the other regular-shaped blocks. An enlarged detail illustrating these conditions is reproduced in Fig. 20 (*a*), in which the transverse beam is shown not hatched (although it is obviously a part of the section), in order to avoid obscuring other details.

If the sizes of the notches are assumed to decrease gradually from the end section toward the region where the sign of notches changes, as suggested in the various illustrations in the paper, conditions are still more exaggerated. Suppose, for instance, that the four notches have the widths of 10, $7\frac{1}{2}$, 5, and $2\frac{1}{2}°$, respectively; then the foregoing relation will have the form: $10\times 45 + 7\frac{1}{2} \times 90 + 5 \times 135 + 2\frac{1}{2} \times 170 + 205\,x = (2\frac{1}{2} + 5 + 7\frac{1}{2} + 10 + x)\,200$, or, $x = 555$ degrees.

Evidently, such values are of no use. They may be reduced, to a certain extent, by: (1) Increasing the available space between the two radial vectors, by increasing either the "middle distances" or the thickness of the arch; and (2) reducing the total opening of the notches either by decreasing their number, or the opening of each notch. As already noted, these provisions can be utilized only to a certain extent.

In fact, an increase of the values of Y'_1 and Y'_2 implies a reduction in the total opening of the position notches; that is, those in the extrados. The maximum of space for the compensating notches between the radial vectors corresponding to Y'_1 and Y'_2 is realized when the former is the ordinate corresponding to the down-stream point of the symmetry section; but in this case there is, of course, no space for any positive notches to be compensated. At any rate, a superficial glance at the arch in Fig. 20 indicates that by increasing the "middle distances," conditions can not be improved very much.

They can be improved to some extent by increasing the thickness of the arch; but to increase the thickness of an arch just for the purpose of a better arrangement of the notches, is a waste of material. There are thin arches, such as the one shown in Fig. 20. The top arch in the author's model, in Fig. 11, is only slightly thicker than the arch now being discussed. Furthermore, a close study of geometrical conditions in thin and thick arches reveals that in reality not much is gained by thickening the arches because almost all the additional space between the two radial vectors must be utilized in order to accommodate the two balancing notches.

It would not be possible to construct notches with openings that are too small. On the other hand, to reduce the number of notches involves serious difficulties. Fig. 21 represents a central notch opened about 5 degrees. When the notches are closed, the faces of the central notch become vertical, while,

for reasons of symmetry, the assumed thrust, T, will be horizontal. In other words, the thrust acts normal to the face of the notch, and not tangential to the neutral axis of the arch. This fact implies secondary stresses in addition to those depending on the usual stress régime in cylinders. To re-establish the latter régime, a perfect cylindrical segment may be secured by cutting off the notch, by means of a radius passing through its up-stream

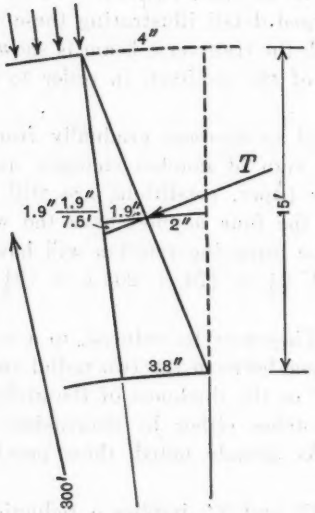

FIG. 21

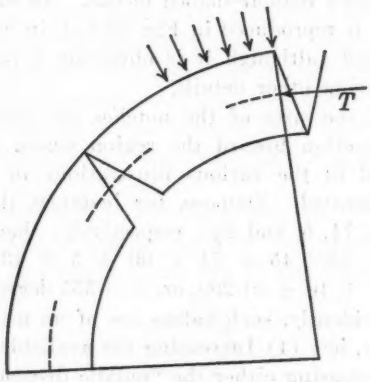

FIG. 22

corner. The original thrust, T, acting in the neutral axis of the notch section must be resolved into the same thrust, T, acting at the center of the new section tangential to its neutral axis, and into a vectorial remainder, which represents the secondary stresses.

Utilizing the data given in Fig. 21, it is easy to verify the statement that the secondary stresses will originate from a bending moment of the magnitude, $\frac{1.9 \text{ in.} \times 1.9 \text{ in.}}{7.5 \text{ ft}} T = 0.05T$, from a vertical force of the magnitude, $T \sin \gamma = 0.0217T$, and from a horizontal force equal to the difference of T and of $T \cos \gamma$ (that is, $T - T \cos \gamma = 0.000235T$). All of them yield clockwise moments for any section, to the left of the notch.

Assume, now that the next notch is on the bisector of the left half arch, and that its horizontal and vertical distances from the points of application of the foregoing forces are 106 ft and 44 ft, respectively (Fig. 22). Then, the resulting bending moment will be:

$$M = 0.04T + 0.0217 \times 106 \times 12T + 0.000235 \times 44 \times 12T = 27.6T$$

This bending moment will cause secondary stresses, in the neighborhood of the bisector, in addition to the uniform compression between the two notches, created by the presence of T (acting tangentially on the radial section as

corrected at the left side of the central notch) and p, the uniform water pressure.

The relative magnitude of cylinder and maximum secondary stresses is: $\frac{T}{t} : \frac{6 \times 27.6 \times T}{t^2} = 1 : \frac{165.6}{180} = 100 : \pm 92$. It is seen that, at least in this case, the extreme stresses in that particular section are $1.92\,\frac{T}{t}$ and $0.08\,\frac{T}{t}$, which are quite different from the assumed uniform stress distribution of the intensity of $\frac{T}{t}$.

Another trouble caused by blocks that are too long is the nullification of Equations (67) and (68), which have been derived on the assumption that the lengths of the blocks as well as the widths of the notches, are infinitesimal. Blocks of substantial lengths and notches of substantial widths change those formulas substantially. The problem presents two alternatives at this point. The notches must be appreciable, or they cannot be built. On the other hand, a reasonably large number of them must be built, or the assumption of a uniform compression in the arch is misleading, and the important formulas, Equations (67) and (68), lose their validity. Then, again, if there is a reasonably large number of notches having reasonably large widths, the region in which the notches pass from opening inward to opening outward (reserved for the two large compensating notches) may not be sufficient to accommodate both these large notches.

None of these difficulties is brought out by the author. Judging from the paper alone, it seems that there is practically no limitation to the arbitrary selection of the positions and widths of the notches; but, in reality, the procedure of selection is exacting because: (1) The total opening of the positive and negative notches must have the same value; and (2) since the difference of their static moments with respect to the chord is practically negligible, their lever arms then have practically equal lengths.

The author gives definite formulas for the widths of the notches, depending upon the radial deflections. This seems simple enough, but it is merely a shifting of the difficulties. The fact is that new troubles arise in selecting a compatible curve of deflections that are avoided in selecting a possible set of notches. Arbitrary deflection curves will not suffice.

If a certain deflection curve, D_R, is selected, it is necessary to determine whether it is compatible with the limitations set by Equations (69) and (113). To do so, the deflection curve selected must first be substituted in Equation (112) for D_R, and then Equation (112) must be solved. The function, β, thus obtained must be substituted into Equations (69) and (113). As a rule it will not satisfy these equations and another selection for D_R must be made, and substituted in Equation (112), in order to determine another function, β, which must again be substituted in Equations (69) and (113), and so on, until the proper value is determined. The operations involved are rather forbid-

ding. Conditions are made more difficult, if the tangential instead of the radial deflections are to be selected.

At this point, the author's selection of the radial deflection according to Equation (78) will be discussed. Substituting the value of β from Equation (79) in Equations (69) and (113), the following relations will result:

$$\int_0^s \frac{2}{t}\left(\frac{\gamma}{R^2} - C\right) ds = 0 \ldots\ldots \ldots\ldots\ldots\ldots (123)$$

and,

$$\int_0^s \frac{2}{t}\left(\frac{\gamma}{R^2} - C\right) Y\, ds = C\, L \ldots\ldots\ldots\ldots\ldots\ldots (124)$$

in which, L denotes the length of the chord, and C is the sign for $\frac{IT}{Et} - c_T T_T$. In another form:

$$2\,\frac{\gamma}{t}\int_0^s \frac{1}{R^2}\, ds = 2\,\frac{C}{t}\int_0^s ds$$

and,

$$2\,\frac{\gamma}{t}\int_0^s \frac{1}{R^2}\, Y\, ds = 2\,\frac{C}{t}\int_0^s Y\, ds + C\, L$$

Dividing Equation (124) by Equation (123):

$$\int_0^s \frac{1}{R^2}\, Y\, ds : \int_0^s \frac{1}{R^2}\, ds = \int_0^s Y\, ds : \int_0^s ds + L\, t : 2 \int_0^s ds \ldots (125)$$

Considering once more the top arch in Fig. 11 and making use of its dimensions, the second term of the right member of Equation (6) will have the value of about $0.4t$.

The first term of the right-hand member represents the distance of the center of gravity of the arch from the chord. Now, if the second term of the right-hand member were equal to zero, or if it were negligible, the relation would express the fact that the distribution of the function, $\frac{1}{R^2}$, along the arch could not be uniform between the end section and the center section. A function, $\frac{1}{R^2}$, having constant values along the entire arch would be compatible with the formula; functions other than constant, if compatible with Equation (115), must increase and decrease from the end section to the center section. Arches of such characteristics are not as a rule, used in dam engineering; radii of curvature usually decrease or increase uniformly, from end section to center section. A comparison of the value of that term with the distance of the center of gravity of the arch from the chord shows that Equation (115) admits only such uniform functions, $\frac{1}{R^2}$, for which the minimum value at the end section and the maximum value at the center section are nearly equal. Generally, this is not the case. The conditions of curvature

involved are, at any rate, extremely different in the author's model arch (Fig. 11). For that arch, the left-hand member of Equation (115) (which represents the distance, from the chord, of the center of gravity of the hypothetical weights, $\frac{1}{R^2}$, situated on the arch) has a much larger value than the right-hand member, which represents the distance, from the chord, of the center of gravity of the arch itself, increased by the insignificant quantity, $0.4 \times t$. Thus, admitting only arches with radii of curvature ever increasing from center to end sections, Equation (125) would be impossible, and, hence, the author's choice of D_R according to Equation (78) appears to be erroneous. The writer modifies his statement with the words, "appears to be", because it is not feasible to segregate the influence of the assumption, Equation (78), from that of Equation (74) which is in either case erroneous as previously explained.

One further remark by the author should be corrected, namely, the statement near the end of Section V claiming that "the sizes of the V-shaped joints that are computed in accordance with the foregoing formulas represent the minimum necessary to compensate the arch." This statement is misleading. The formulas to which the author refers are manifestly those representing the proper solution of Equations (67) and (68), or Equations (82) or (85); but there is no evidence of the validity of the author's contention. The values of β as given by the foregoing formulas are correlated to a selected set of, say, radial deflections but they do not represent the minimum notches necessary to compensate the arch. There are no smaller or larger notches compatible with that selected set of radial deflections; there is only one set of notches.

The writer candidly confesses that he was not able to grasp the author's ideas and his explanations pertaining to the process of compensating arch dams (Section XI and Section VII, following Equation (91). It is regrettable that in a paper entitled "The Compensated Arch Dam," only half a page (Section VII), deals with the most difficult phase of this problem, and that only half a page (Section XI) is devoted to a problem that is the dream of dam engineers, namely, the reproduction of a dam having the most economic shape in a given canyon section. On the other hand, the author devotes about 36% of his paper to general aspects of arch dam design, which are common knowledge among those whom he may reasonably expect to understand his theory of the compensation of arch slices and to pass judgment upon its usefulness. This is a redactional inconsistency, which should have been avoided.

To pass a final judgment on the paper, the writer is pleased to give full credit to the author for the original conception and the searching treatment of the compensated arch. However, he could not help feeling disillusioned at the end of that treatment. The entire analysis is based on the assumption that the lengths of the blocks are infinitesimal. This implies that the widths of the notches are infinitesimal values of the second order. With commendable accuracy, the author thought it necessary to distinguish between ϕ and

$\beta = \left(I - \frac{T}{Et} - c_T T_T\right) \phi = 0.99995\ \phi$, thus introducing the difference of $0.00005\ \phi$, which is an infinitesimal of the third order. After such painstaking analytical research of the widths of the notches, he concludes (near the end of Section V):

"As far as the actual construction of the notches is concerned, one of the simplest methods is to place the concrete in alternate blocks and to provide the V-shaped joints, by means of elastic fillers placed between the adjoining blocks. During the time of loading the arch, the V-shaped joints will close and compress the fillers flat or squeeze them out."

However, the paper does not contain any information, nor does it attempt to analyze the following questions: How will the joint filler affect the action? How much will the filler material compress, or squeeze out? What will be the effect of such compression or squeezing on the notches and the arch? Hence, the following alternatives: Either the author has, or has not, developed at least a rough quantitative analysis of the notches with fillers. If he has, he should have published that analysis of general validity instead of the one of particular validity now being discussed. Otherwise, he will have to handle the notches and fillers in the so-called "practical" way, in which case the searching theory of the ideal notches having no fillers looks very much like hair-splitting—up to third infinitesimals.

So much for isolated arches. As to the proposed method of compensating the entire dam, the writer ventures to present the following opinion: The method proposed in the paper is incomparably more complex than the trial-load method which, certainly, requires considerable labor. Recalling the great difficulties of selecting sets of deflections of isolated arches compatible with Equations (69) and (113), and appraising the formidable difficulties of such selections in arched dams, in which, in addition to the radial load, twisting moments and tangential shearing stresses must be dealt with, the labor required for the analysis of the dam as a whole seems to be prohibitive.

The writer objects to the complexity of the designing process only in a secondary way. His main objections to the type of dam proposed are based on the following:

(1) The uncertainty of its mechanical behavior;

(2) The impossibility of reconciling the practical problems of construction with the ideal assumptions of the paper; and

(3) The artificiality of the concept, implying the dissolution of a solid body of a structure into a great number of loose pieces. This involves an abandonment of the reserve strength that engineers properly associate with the idea of a monolithic mass structure, and there seems to be no adaquate advantage in it.

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DISCUSSIONS

IMPROVEMENT OF HUDSON RIVER BY NARROWING THE NAVIGABLE FAIRWAY

Discussion

By LYNNE J. BEVAN, M. AM. SOC. C. E.

LYNNE J. BEVAN,[9] M. AM. SOC. C. E. (by letter).[9a]—The author appreciates the constructive criticism presented in the discussion. General Brown's suggestion that ships' propellers may be a material factor in scour is interesting. Intense agitation near the river bottom tends to disturb the stability of the bed and gives Nature a better chance to operate. If propeller agitation is a considerable factor in scour, is it not reasonable to expect the maximum scour along the line most used by the larger steamers, and in the areas agitated by their warping-in operations? From data in their offices, the Engineers of the U. S. War Department might be able to contribute something definite in this connection. From a casual inspection of the sections plotted, however, the scour appears to have occurred fully as much, if not to an even greater extent, along the New Jersey side as along the more intensely navigated Manhattan side.

General Brown's comments concerning the commerce raises questions outside the scope of the paper. It is suggested, however, that the 119 collisions between 1926 and 1930, inclusive, were in a large measure caused not by congestion—which is comparatively non-existent in the Hudson River—but either by poor visibility or by negligence in navigation. Who will argue that the 119 collisions are more than would have occurred if the width of the river at the Lackawanna and Erie Ferries had not been reduced since 1767 from 6 000 ft to 3 000 ft?

In connection with the discussion by Mr. Moss, the writer would like to explain that his original interest in this hydraulic problem as applied to the Hudson River was aroused in 1931 when he was retained in behalf of an applicant for pierhead extension and discovered that the Secretaries of War

NOTE.—The paper by Lynne J. Bevan, M. Am. Soc. C. E., was published in May, 1932, *Proceedings*, Part 1. Discussion on the paper has appeared in *Proceedings*, as follows: October, 1932, by Messrs. William B. Moss, Lytle Brown, and H. A. Marmer.

[9] Cons. Engr., New York, N. Y.

[9a] Received by the Secretary January 27, 1933.

in 1913 and in 1920 had denied similar applications with memoranda stating that as a result of pier extensions, "the speed of the tide has already been increased over 60 per cent." Consequently, the intent in the paper under discussion was to bring consideration of the Hudson River problem out into the realm of sound and broad hydraulic theory. However, Mr. Moss' assertions of "inaccuracies and errors in assumptions and basic data" used, should not pass unchallenged.

There may be some inadvertent errors, but most of the alleged errors were deliberate as they were in the direction of conservatism in the main thesis. The first "error in assumption," namely, that the effective cross-section is included between the pierhead lines, was deliberately incorporated since this assumption is less favorable to the thesis and hence makes the argument less susceptible to criticism of details. This holds whether Mr. Moss' contention refers to a pier extending westerly from Manhattan in 1855 to the 1930 pierhead line, or simply to tidal water flowing behind later pierhead lines. In general, if the effective cross-section area is increased to admit the tidal flow back of the pierhead line, the areas in Table 4, Columns (4) and (6), are increased more than those in Columns (2) and (3), and thus the mean velocity of the tidal currents is still further reduced in comparison with the velocities of earlier years.

As to the second "error in assumption," that is, "that the charts show soundings of the date of publication," the writer, of course, made no assumption that the soundings shown on the 1930 chart were made in 1930. He simply used the 1930 chart as the best information available concerning depths existing in 1930—as do navigators, the Army engineers, the U. S. Coast and Geodetic Survey, and other interested parties. Through the courtesy of the late Robert Lee Faris, M. Am. Soc. C. E., the author was furnished, in May, 1931, with copies of the 1930 charts with the dates of the surveys indicated thereon. The part of the river from Section 4 to Section 9 shows 52.8% of the area surveyed subsequent to 1910. It is regrettable, of course, for the purpose of this discussion, that absolutely complete sounding data for each date considered are not available; but the absence of recent soundings in a substantial part of the river bed may be an indication, perhaps, that shoaling has not occurred sufficiently to interfere seriously with navigation.

Table 6 shows quite generally a deepening of the fairway. The only shoaling in excess of the nominal amount of 1.0 ft has occurred in grid areas largely behind the pierhead lines. In the case of the grid, 12-*A*, it is entirely behind the bulk-head line! The mean depth between the 1930 pierhead lines of the eighteen sections of Table 4 was 35.9 ft in 1855, and 42.2 ft by the 1930 charts. This is an increase of 17.6%, of which 15.3% is due to scouring by Nature and 2.3% to dredging.

As to Table 7, it has only to be pointed out that, while at the exact section used structures and fill did not reach out to the established pierhead lines, a short distance above and below the exact section, structures are shown substantially out to the pierhead line. For instance, the area of Section 3 in

1930 is presumably computed, in Table 7, to the end of the Pennsylvania Ferry racks on the New Jersey side which are more than 300 ft inside the pierhead line; but the flow-control pier at the north end of the ferry racks extends almost to the pierhead line. In 1855, the irregularity of structures was, doubtless, greater than it is to-day.

Mr. Moss correctly notes that in Figs. 2 and 4 the 1914 and 1897 pierhead lines coincide, and that this is legally in error. The 1914 line was shown 100 ft out from the 1897 line for the reason that Pier 54 below, and Piers 59 and 60 above, Section 6 had been built out 100 ft under temporary permit, and this outer line was used as the conservative one, and to avoid the criticism just applied to Table 7.

Mr. Marmer's discussion is greatly appreciated as it comes from many years of competent study of tides. Had the writer had the benefit of frequent consultation with Mr. Marmer during the preparation of the paper, he might have endeavored to apply corrections due to the considerable decrease in tidal range from 1885 to 1910 and the partial return since 1910. The change during the entire period from 1855 to 1930 shows 10% decrease in tidal range, and, hence, aproximately the same decrease in tidal current quantities, and thus, in general, adds a factor of safety to the conclusions in the writer's paper.

As to the effect of the run-off of the Hudson River drainage area of 13 400 sq miles, the writer wishes to make reference to the comprehensive paper on "Tidal Phenomena in New York Harbor," by H. de B. Parsons, M. Am. Soc. C. E. This work provides comparisons of land-water discharge volumes with tide-water volumes, as follows:[10]

"The mean yearly discharge of land-water run-off past Battery Place for 6 lunar hours is 543 600 000 cu. ft. * * * The average ebb flow will be the mean given above plus one-half of this net discharge, and the average flood volume will be the mean less one-half of the net discharge, or:

"Average volume of ebb current, 6 990 600 000 cu. ft.
"Average volume of flood current, 5 903 400 000 cu. ft."

Thus, land-water drainage is only 7.8% of the ebb tide.

Since 1855 the width of the fairway between pierhead lines has been decreased roughly 12.3%, so that the percentage increase in the ebb-current velocity near the surface might be 0.96% under Mr. Marmer's suggestion of substantial separation of fresh and salt water. The effect of this increase, as it applies to deep-draft vessels, is largely offset by the corresponding decrease in current velocities at greater depths.

However, the very fact that average depths have been increased by Nature in compensation of the narrowing of the river is suggestive that an increase of ebb-current velocities has taken place near the river bed. This idea conforms with the general hydraulic experience recalled in the second paragraph of the paper, that narrowing a channel tends to make velocity more uniform from the surface to the bottom. If this is so, the probabilities are that surface velocities have actually decreased because, as was shown in the paper, the mean velocity has decreased.

[10] *Transactions*, Am. Soc. C. E., LXXVI (1913), p. 2038.

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DISCUSSIONS

APPLICATION OF DURATION CURVES TO HYDRO-ELECTRIC STUDIES

Discussion

By G. H. HICKOX AND G. O. WESSENAUER, JUNIORS, AM. SOC. C. E.

G. H. HICKOX[15] and G. O. WESSENAUER,[16] JUNIORS, AM. SOC. C. E. (by letter)[16a]—The writers have attempted to present in as simple a fashion as possible, the basic ideas underlying their application of duration curves to hydro-electric studies. It was assumed that the paper would be of interest chiefly to those having had experience with similar problems. For this reason, analyses of the familiar mass curve and hydrograph methods were omitted, as well as the economic features inherent in all such problems.

Such factors as weekly variation of load, seasonal variation of load, variation of head due to reservoir draw-down, variations of turbine efficiency with head, evaporation from reservoir, pondage, regulation of flow by diversion dams, reduction of head due to back-water, plant load factor, system load factor, and other variables which occur in the practical application of any method of estimating power, and which, while important, were not essential in explaining the method devised, were purposely omitted. The same factors arise in connection with the use of the mass curve, and one who has had experience in dealing with them for use with such studies should have no difficulty in applying them to the duration curves.

A few of the discussers appear to think that the writers advocated the use of the duration curve for all the cases described. This does not necessarily follow. Their aim was to present the essential elements of an idea which is capable of endless variation. The illustrations selected were purposely simple ones, in order that the principles involved might be described

NOTE.—The paper by G. H. Hickox and G. O. Wessenauer, Juniors, Am. Soc. C. E. was published in May, 1932, *Proceedings*, Part 1. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1932, by Messrs. Paul F. Kruse, Donald H. Mattern, James E. Stewart, Don Johnstone, L. T. Guy, and Joel D. Justin; November, 1932, by C. Maxwell Stanley, Jun. Am. Soc. C. E.; and January, 1933, by Messrs. F. Knapp and Robert F. Ewald.

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[16] Asst. Engr., The West Penn Elec. Co., Pittsburgh, Pa.

[16a] Received by the Secretary January 7, 1933.

without the fog of a multitude of extraneous factors. Any problem in power rates may be solved by the use of daily records (the hydrograph method), a great many problems by the mass curve, and a great many by the duration curve. Each has its own field of usefulness, and the choice of a method depends on the conditions of the problem to be solved. The mass curve is applicable to the simpler cases, and for these may be most desirable. For problems involving load factor and a number of plants the mass curve becomes less useful and the duration curve more useful. The mass curve method falls down when the capacity at the run-of-river plant is limited, because it makes no provision for evaluating the energy lost when water must be by-passed at the lower plant due to the lack of capacity. The evaluation of this loss converts a mass curve study into what is essentially a hydrograph study by days. Probably the most useful function of the mass curve is as an indicator of the critical period for reservoir operation. The duration curve method as described fails only when the flows at the various sites are unduly dissimilar in point of distribution.

If the flow at two or more points under consideration is so dissimilar as to throw doubt on the results obtained by the duration curve, the effect of the dissimilarity can be evaluated by making a study of a simple set-up by the hydrograph method and comparing the results with those obtained by the duration curve. If the error is within permissible limits for the simple case, the duration curve method may be used safely for the more complicated cases.

The second basic assumption, that flows at one plant are immediately available at the next down-stream plant, is not very important. In most cases, plants are close enough together and are provided with sufficient pondage so that water can be drawn from the lower ponds with positive assurance that it will be replaced within a short time by water liberated at the plants above.

The third basic assumption apparently was loosely stated. As used in the paper, "unlimited storage" is merely a relative term. Presumably, an investigation of a group of proposed developments would involve the determination of the primary energy rate, plant capacities, and reservoir capacities. Different values would be obtained by varying the length of the reservoir cycle considered. The results then would be analyzed from an economic and practical standpoint (including topographical limitations of storage). Therefore, if the period selected involves an impossible amount of storage capacity, the obvious procedure is to shorten the period. This point was brought out in the paper under the heading, "General Discussion", and was also emphasized in Mr. Stanley's excellent discussion. Mr. Johnstone's statement that the third basic assumption "causes the greatest limitation of usefulness," is thus seen to be incorrect.

Mr. Johnstone's belief that the mass curve is applicable to all problems which can be solved by the duration curve and that it will yield the same results with less effort is erroneous. The duration curve method described was developed for situations in which the mass curve was not readily appli-

cable; namely, the combination of a proposed storage plant with an existing run-of-river plant with a fixed capacity. The combined plants were to operate with a load factor of 30 per cent. The capacity of the run-of-river plant was thus frequently exceeded when water was drawn from storage to produce power at the up-stream plant. The evaluation of power thus lost converts the mass curve study into a daily hydrograph study requiring weeks of time, while the evaluation can be made with the duration curves in as many days.

Mr. Johnstone also questions the layman's understanding of the duration curve. It is not necessary to explain it if used only for preliminary studies. Final estimates should be made on the basis of hydrograph studies. However, the writers have found no more difficulty in explaining that an area means a definite quantity of power than they have in explaining the mass curve in which the reservoir is being depleted while the flow ordinate climbs steadily upward. Mr. Johnstone's comment on determination of storage capacity in Case 1 is well made.

The preceding discussion should be in part an answer to Mr. Mattern's query. It is necessary to reduce the flows at the various sites to a common denominator which is affected by head and efficiency. Since power output is the ultimate object of such studies, it was found convenient to use the kilowatt as the unit. Mr. Johnstone's equivalent flow, further modified to include efficiencies at each head, might be used, but this would still require ultimate conversion to kilowatts, and, in the meantime, would not give as clear a picture of the output from each plant.

Mr. Knapp seems to feel that the power-energy curve proposed by Mr. Hutchinson has a greater applicability than the duration curve method. Mr. Hutchinson's power-energy curve is derived from a system load curve and is a convenient method of dealing with the load factor of parts of the system load. The power-energy curve is a useful tool in determining the effect of load factor, but it does not offer a complete analysis of the possibilities for development of a stream. Mr. Hutchinson used an integrated duration curve in his studies,[17] but his use was confined to a study of a run-of-river plant with a steam auxiliary plant.

Mr. Hutchinson's method avoids using data for one reservoir cycle by ignoring the effect of storage. Duration curve studies have been commonly made on the basis of the average flow over the entire period of record, leading to fantastically high rates of power output. As far as the writers know, the only method of determining the reliable rate of output is by the use of data covering one reservoir cycle with a concurrent investigation of reservoir capacity to see if there is reserve enough to supply the deficiency. Estimates based on the lowest calendar year of record may also be grossly misleading in this respect.

Mr. Ewald mentions a modification of the mass curve which is cumulative about the mean discharge, and Mr. Guy calls attention to a similar curve. The writers agree that this form of mass curve is the most convenient one when using that method.

[17] "The Economical Capacity of a Combined Hydro-Electric and Steam Power Plant," by Cary T. Hutchinson, *Transactions*, Am. Inst. E. E., Vol. XXXIII, p. 155.

Mr. Kruse's suggestion that the term, "flow duration curves," as used in the paper under the heading, "Advantages," be changed to "output-duration curves,"[2] is well put. The paper states specifically that flow duration curves need not be constructed.

Mr. Justin, Mr. Knapp, and Mr. Ewald stress the importance of economic considerations. This factor can scarcely be over-emphasized. At the present cost of steam power development, a hydro-electric plant must be able to show a definite contribution to the primary energy requirement of the system in order to justify itself. No factor that will affect the amount or cost of output should be overlooked.

The writers believe that the duration curve method permits a more exhaustive analysis of the possibilities of a group of developments at a given cost than is possible with any other known method without loss in accuracy. It is believed that engineers who familiarize themselves with this method will continue to use it.

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DISCUSSIONS

FORESTS AND STREAM FLOW

Discussion

BY MESSRS. W. C. LOWDERMILK, RHODES E. RULE, AND ROBERT E. KENNEDY

W. C. LOWDERMILK,[50] ESQ. (by letter).[50a]—Questions are raised in this interesting paper concerning the influence of a mantle of forest vegetation, primarily on the yield of summer flow from water-sheds. The general conclusions, if tenable and if followed to logical ends, involve far-reaching consequences. The paper is one of the most important of recent contributions to the subject of forests and stream flow and requires scrutiny into methods used, analyses, and interpretation of data.

Conclusions are based on two water-shed studies, in diverse regions, under widely different climates. The Wagonwheel Gap area lies above an altitude of 9 000 ft in the snow belt of the Rocky Mountain region, within a continental climate in which annual precipitation is characterized by two approximately equal maxima, one as snow in winter, and the other as thunderstorms, or convectional storms, in summer. In this region the regimen of flow of the Rio Grande to which the Water-Sheds *A* and *B* are tributary, is well regulated, showing by channel conditions a minimum of sedimentation and irregularity of flow. The Southern California area, including Big Santa Anita and Fish Creek drainages, on the other hand, lies between elevations of 1 500 and 5 000 ft, within the coastal range of mountains, and within a typical Mediterranean climate in which precipitation is characterized by one maximum in winter and a long hot rainless summer. Regimens of stream flow show characteristic semi-arid type of irregularity with well developed outwash fans of coarse detrital material and extensive detrital filled basins.

NOTE.—The paper by W. G. Hoyt, M. Am. Soc. C. E., and H. C. Troxell, Assoc. M. Am. Soc. C. E., was presented at the Anuual Convention, Yellowstone National Park, Wyoming, July 6, 1932, and was published in August, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1932, by C. G. Bates, Esq.; November, 1932, by Messrs. J. E. Willoughby, and A. L. Sonderegger; December, 1932, by Messrs. J. C. Stevens, Harry F. Blaney, Daniel W. Mead, Ralph R. Randell, H. K. Barrows, Donald M. Baker, Ralph A. Smead, and George H. Cecil; and February, 1933, by Messrs. C. W. Sopp, and W. B. Rowe.

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[50a] Received by the Secretary January 3, 1933.

The Wagonwheel Gap study was conducted with great accuracy with respect to measurement of precipitation over water-sheds and environmental climatic factors and stream flow, whereas the Southern California study was an incident in stream gauging over a period of years along the mountain front of Southern California. No provision was made for an accurate determination of catch of water-sheds in question, nor of other climatic factors. Precipitation records of Mt. Wilson, an eminence on the west margin of the Big Santa Anita drainage area, are introduced with the assumption that precipitation fell at uniform ratios over the two drainages under observation. While the use of the unburned Big Santa Anita Water-Shed as an integrator of climatic factors is sound, it can serve legitimately only when total catch in precipitation is determined by measurements for the two drainages. Variation in rainfall from place to place within cyclonic disturbances in Southern California as shown in Table 18 is sufficient to impose a serious limitation upon

TABLE 18.—Annual Precipitation of Selected Stations, Showing Irregular Variations of Catch, Southern California

Item (1)	Hydrologic years * (2)	Mt. Wilson (3)	Big Santa Anita (4)	San Gabriel Intake (5)	Pasadena (6)	Monrovia (7)	San Antonio (8)	San Antonio Intake (9)
1.....	1918–1919	21.98	.....	17.74	14.60	16.09	19.99	19.20
2.....	1919–1920	27.17	.....	25.55	16.77	19.69	30.43	27.98
3.....	1920–1921	34.28	33.95	26.30	19.59	24.41	34.58	32.90
4.....	1921–1922	60.26	57.33	57.58	29.36	31.73	55.35	54.18
5.....	1922–1923	24.48	26.71	23.60	14.01	17.16	27.02	27.66
6.....	1923–1924	16.76	16.05	14.20	8.94	11.87	19.46	19.76
7.....	1924–1925	21.91	21.15	16.95	12.88	16.34	19.89	19.36
8.....	1925–1926	36.26	39.60	31.42	22.44	22.21	37.01	37.34
9.....	1926–1927	33.37	39.17	34.19	25.13	27.24	38.67	38.51
10.....	1927–1928	19.78	18.46	15.26	13.73	12.84	18.40	20.41
11.....	1928–1929	20.68	23.03	23.10	16.42	17.57	28.10	.....
12.....	1929–1930	23.00	22.68	.....	15.79	18.33	27.19	.....
13.....	Average....	28.52	29.81	25.99	17.78	19.03	29.67	29.72

* Water-year, or hydrologic year, October 1 to September 30.

the validity of the conclusions drawn from the study. Compare, for example, Items 8 and 9, using the catch of Big Santa Anita (Column (4)) as starting points. It will be noted that variations of 3 to 5 in. occur, in opposite directions.

The authors' description of vegetation for the Southern California area is inadequate in comparison with that of the Wagonwheel Gap study. The inadequacy of the description of vegetative cover is apparent in references to the paper compared with Fig. 13 and Table 19, in which data collected from twenty-five sample plots, under the direction of Mr. A. E. Wieslander, for the State Type Map, were generously supplied, and are summarized. The vegetative survey was made in 1928, four years following the burning of Fish Canyon. Dominant vegetation was mapped in sample plots shown on the map, and this accounts for absence of reference to subordinate occurrence of yucca and other species named by the authors. Riparian, or canyon-bottom, vegetation is not included in Table 19. Field sample plots are indicated by numbered circles in Fig. 13. Type lines were drawn on base U. S.

FIG. 13.—BIG SANTA ANITA AND FISH CREEK DRAINAGES, SHOWING COVERAGE BY VEGETATIVE TYPES AND LOCATION OF SAMPLE PLOTS OF 100 MILACRES, OR 0.1 ACRE EACH.

TABLE 19.—Composition of Chaparral Vegetation in Fish and Big Santa Anita Canyons

Vegetation		Coverage (Average Percentage)*	
Common name	Botanical name	Big Santa Anita drainage	Fish Creek drainage
Chamise	*Adenostoma fasciculatum*	20.00	10.0
Manzanita	*Arctostaphylos canescens*	4.5	6.0+
California sage	*Artemisia californica*	Trace	....
Whitebark soapbloom	*Ceanothus divaricatus*	8.0	7.3+
White hornbrush	*Ceanothus crassifolius*	13.0	1.2
Ceanothus	*Ceanothus oliganthus*	4.8	3.7
Birchleaf Mt. Mahogany	*Cercocarpus betuloides*	7.6	4.0
Wild buckwheat	*Eriogonum fasciculatum*	0.9	2.4
Grass	Grass	Trace	2.0
Christmas berry	*Photinia arbutifolia*	4.5	3.5
Hollyleaf cherry	*Prunus ilicifolia*	.....	4.6
California. scrub oak	*Quercus dumosa*	8.0	11.0
Scrub interior live oak	*Quercus wislizenii var. frutescens*	8.0	5.6
Scrub canyon live oak	*Quercus chrysolepis nana*	.....	0.6
Deerweed	*Lotus scoparius*	.....	3.7
Black sage	*Salvia mellifera*	4.0	2.0
White sage	*Salvia apiana*	2.5	3.6
Red-berry	*Rhamnus crocea var. ilicifolia*	2.8	1.4
Sugar bush	*Rhus ovata*	4.0	5.5
Laurel sumac	*Rhus laurina*	1.5	....
Miscellaneous species	.....	1.4	4.9
Bare surfaces	.....	4.5	17.0
Litter, depth	.....	0.5 to 3 in.	Trace
Total	.....	100.0 unburned	100.0 burned over 1924

*Coverage refers to dominant vegetation; subordinate species under dominant cover are not listed.

Geological Survey quadrangle sheets from occupied points, checked by detailed sampling in plots.

Apart from the description of vegetation on slopes, the paper by Messrs. Hoyt and Troxell is entirely misleading with respect to the condition of the canyon-bottom growth of alders, willows, sycamore, poplar, and oaks of the woodland type. It is stated by the authors that this growth was protected from the fire and so it was below the falls in Fish Canyon. Above the falls, however, the mesophytic sub-irrigated canyon-bottom growth was entirely killed, as checked by the writer and recorded by numerous photographs. This discrepancy of fact and description imposes another important limitation upon the conclusions regarding summer flow which will be referred to again.

Absence of treatment of geological structure and soil formation imposes another limitation upon the study as a corroboration of the findings in the Wagonwheel Gap. Plotting of run-off against rainfall shows closer agreement between the catches measured in Big Santa Anita Canyon than at Mt. Wilson, with a further indication that Big Santa Anita displays greater storage than Fish Creek under heavier intensity of rain when both areas were burned. The thin soils on the slopes of young mountains in Southern California offer comparatively small storage in comparison with deep soils of more gentle topography.

Before examining the principal question raised in this paper regarding effect of denudation on summer flow, it may be pointed out that the paper makes two important contributions: (1) It refutes certain important con-

clusions[51] of the late H. M. Chittenden, M. Am. Soc. C. E., cited in the paper; and (2) it indicates that erosion following denudation in Southern California is increased to a marked degree, to which experience and experiment fully attest.

A reference to the baneful influence of Colonel Chittenden's memorable paper is in place here. His conclusion that the "bed of humus" or litter of a forest floor is the chief agent of forest cover in absorption of precipitation is "an outstanding fallacy." Refutations of his conclusions in the discussion appended to his paper, require as much attention as his original paper. More recently the writer's studies[52] have shown that the capacity of a layer of forest litter to absorb water is of small importance in comparison with its function to maintain surface waters clear and the absorption capacity of underlying soil at its maximum. The mantle of soil in the final analysis is the absorbent of precipitation, not the forest. The forest cover, especially the litter which it produces, furnishes conditions which make for maximum possible water and erosion control on water-shed surfaces. It is curious that Colonel Chittenden's paper should continue to have its influence upon textbooks, writings, and opinions in this field of forests and stream flow when the

TABLE 20.—Comparison of Run-Off from Fish Creek Drainage as Mean Daily Discharge and as Summer Flow After Burn.

(Computations on the basis of the authors' data)

Hydrologic year	Annual Discharge, in Inches of Depth			Summer Flow: Fish Creek			
	Big Santa Anita Creek	Fish Creek		Normal, in inches of depth	Percentage of annual	Actual, in inches of depth	Percentage of annual
		Normal	Actual				
1924–1925	1.24	1.07	3.54	0.01	0.90	0.15	4.24
1925–1926	7.77	11.80	14.92	0.05	0.40	0.33	2.21
1926–1927	9.24	13.07	14.65	0.14	1.07	0.46	3.14
1927–1928	1.79	1.79	2.47	0.00	....	0.10	4.05
1928–1929	2.17	2.32	3.01	0.00	....	0.04	1.32
1929–1930	2.26	2.35	3.09	0.01	0.43	0.04	1.29

late George Fillmore Swain, Past-President and Hon. M. Am. Soc. C. E., has presented[53] a much more open-minded statement of the problem and deserves to rank in authority much higher than Colonel Chittenden.

The point of particular moment in the paper by Messrs. Hoyt and Troxell is the effect of deforestation by cutting in the Wagonwheel Gap and the denudation by fire in Fish Creek on summer flow, which is commonly said to be reduced by deforestation. The relative flow is small in comparison with the total annual stream discharge as may be seen in Table 20. The ratio of summer flow and the percentages of increase are within the accuracy of measurement of stream flow in the study in Southern California. The contribu-

[51] "Forests and Reservoirs in Their Relation to Stream Flow, with Particular Reference to Navigable Rivers", by H. M. Chittenden, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXII (1909), p. 245.

[52] "Influence of Forest Litter on Run-Off, Percolation, and Erosion," by W. C. Lowdermilk, *Journal of Forestry*, Vol. XXVII, No. 4, April, 1930.

[53] "Conservation of Water by Storage", by George Fillmore Swain, Past-President and Hon. M. Am. Soc. C. E., Chester S. Lyman Lecture Series, Yale Univ. Press, 1914.

tion becomes not of so much importance in indicating substantial increase in yield of water as in the new and conflicting information it presents concerning a widely accepted belief. From a study of the two areas it appears that the indicated increase in summer flow occurs from two different causes. In the case of Wagonwheel Gap the difference is really of minor importance but can be accountd for by difference in interception of summer rain. Interception of as much as 40% of rainfall has been measured. From 10 to 20% is a conservative figure for a dense high forest cover. Since the increase in summer flow following deforestation in Water-Shed *B* was 9%, this amount may be accounted for by reduced interception of summer rains.

Such a possibility is eliminated in the case of the Southern California study in the absence of summer rainfall. Quite another explanation is required. In another paper[54] the writer has set forth an analysis of factors affecting the yield of water from Southern California water-sheds. The salient points are that the mantle of vegetation subdivides into two distinct types as may be noted in Fig. 13, namely vegetation of drained slopes (chaparral) and vegetation of sub-irrigated canyon bottoms (riparian vegetation). Chaparral vegetation on drained slopes has at its disposal after the end of the rainy season only that supply of moisture retained by the soil of the root zone against the pull of gravity, namely, available moisture between water-holding capacity and wilting point. A fire in summer will not release water from such drained soils. In the succeeding spring re-growth of sprouting vegetation and the great coverage of volunteer annuals may be counted on to extract moisture during the long dry season approximately to the wilting point within the soil occupied by roots of sprouting plants and invaded by new growth. In the event that root zones are not as fully occupied the first season following a fire as prior to a fire an increase of run-off coefficients (other factors remaining identical), may be expected. Such increases, however, under the climatic conditions of Southern California are not to be counted on to an important extent.

Quite another situation obtains with the riparian or canyon-bottom vegetation. It is typically mesophytic vegetation, and has at its disposal during the long hot summer an abundant water supply in canyons of perennial flow, such as Fish Canyon. Large quantities of water are transpired into the air by this vegetation. Accordingly, when a fire kills this riparian vegetation transpiration draft is stopped and increased flow in the stream occurs. Not until the riparian vegetation has returned to its former transpiration capacity will the flow be returned to normal.

Since all riparian vegetation in Fish Creek above the falls (see Fig. 13) was killed in the fire of 1924, it is possible to account for the recorded increase of flow by reduction in transpiration losses of this type of vegetation along about 10 lin miles of main canyon and tributaries. For this reason the use of the Wagonwheel Gap experiment to support the findings from Southern California are brought into question as a scientific contribution.

[54] "Studies of Factors Affecting Yield of Water from Watersheds in Southern California", by W. C. Lowdermilk, Proceedings Conference: Problems in Consumptive Use of Water and Conservation of Rainfall, Water Conservation Committee, Irrig. Div., Am. Soc. C. E., Los Angeles, 1930.

This paper emphasizes the need of a thoroughly executed study in Southern California to answer satisfactorily the question raised as to method, and as to instrumentation and completeness in study of all factors. It must also be recognized that the question of transpiration loss on yield will not be the only basis for management of Southern California water-sheds for maximum beneficial yield of water. Storage in underground detrital-filled basins, control of flood discharges, and protection of flood-control reservoirs from sedimentation must enter into enlightened management of water-shed areas in Southern California.

RHODES E. RULE,[55] ASSOC. M. AM. SOC. C. E. (by letter).[55a]—The most important, and perhaps least understood, factor in the relation existing between vegetal cover and water supply is soil moisture storage. An idea prevails that significant quantities of moisture stored in the soil mantle during the rainy season are released later by slow drainage to surface streams, thus supporting the low-water flow of these streams. It is conceived that a dense brush or forest cover assists this process and thus acts as a conserver of water. Such misconception is due largely to mistaken ideas regarding the extent and velocity of soil moisture movement.

Many long-held concepts of soil moisture habit have fallen under the searching criticism of recent research workers. It is now definitely known that, in the absence of direct capillary connection with the water-table, movement of soil moisture under the influence of film forces is extremely slow. Other forces, probably closely related to the colloidal properties of a soil, tend to resist moisture movement and, as the moisture content of a soil decreases, the resultant force causing moisture migration becomes smaller and approaches zero at what Shaw[56] has termed the "normal moisture capacity" of a soil. At this point of static equilibrium the forces of adhesion and colloidal adsorption are great enough to prevent further movement and subsequent change of position must take place in the vapor phase. A wide range of moisture between this point and the hygroscopic coefficient, hitherto considered to be free to move under capillary forces, must remain in the soil until removed by evaporation or plant transpiration. Slow drainage is impossible after moisture in a soil has been reduced to this point.

In Southern California the native plant life draws heavily on the soil moisture supply during the long dry summers and at the beginning of the rainy season there is a considerable deficiency of moisture in the root zone. Many determinations have shown that the entire root zone is close to the wilting percentage of soil moisture before the first autumn rains. The soil moisture reservoir is empty; it must be filled before general percolation can penetrate below the root zone.

The first rain will penetrate the surface layers of soil and quickly fill them to field capacity to a depth dependent on the soil moisture deficiency, the lower portion of the root zone remaining dry. Immediately, evaporation and plant

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[55a] Received by the Secretary January 3, 1933.

[56] "The Normal Moisture Capacity of Soils," by C. F. Shaw, *Soil Science*, Vol. 23, p. 303.

transpiration activities commence and draw on this accretion of moisture. There is little opportunity for slow capillary adjustment to take place within the root zone. In a relatively short time evapo-transpiration losses reduce the soil moisture content to the "normal moisture capacity," and capillary moisture movement ceases.

Succeeding rains, provided they occur soon enough and in sufficient magnitude, gradually push the moistened area downward, but in the period between rains evaporation and transpiration continue to take their toll. If the total rainfall is heavy enough the root zone may finally be filled to field capacity and when this condition is reached deep percolation can begin. It can probably be safely assumed that most of the water that penetrates to below the root zone will be returned in time to surface streams as ground-water seepage.

However, water that does not escape the root zone during the first quick gravitational drainage period following wetting has a negligible chance of ever passing out of reach of the plant roots. After gravitational movement has ceased, the ensuing capillary adjustment is so exceedingly slow and plant demands are so great that escape is practically impossible in any appreciable quantity. Where, as is the case in vast areas of mountain water-shed, the root zone comprises the total depth of the soil mantle, storage within the mantle for later release to streams is impossible except in negligible quantities.

Certainly all ground storage, aside from the permanent ground-water body, which can be classed as possible of future capture by human agencies is of a deep-seated nature, that is, below the root zone. It consists of water in transit under conditions of excessive application. It is water in excess of equilibrium capillary capacity that is working its way down to the water-table. (Equilibrium capillary capacity is the point of soil-moisture equilibrium at which the forces of gravity and capillarity are in exact balance. It is the limiting condition approached in a soil which has been saturated from the ground surface to the water-table and then permitted to drain for an indefinite period without losses by evaporation or transpiration. Thus, it is the limiting moisture content of that part of the lithosphere lying between the lower horizon of the root zone and the water-table in a region periodically subject to deep penetration of precipitation to below the root zone. No water can reach the water-table until the moisture contents of all higher levels of soil or rock along the course of percolation have been raised to equilibrium capillary capacity.)

Where the water-table is in bed-rock, this storage is the water contained in the soil below the root zone in excess of equilibrium capillary capacity, plus the water contained in rock joints in excess of the equilibrium capillary capacity of the joints. The writer's observations of mountain springs and in deep tunnels lead to the conclusion that in mountain water-sheds deep-seated open joints are the principal transmission channels for waters that have escaped the root zone. This is contrary to the opinion sometimes expressed that the moisture that eventually feeds surface streams must percolate along or above the surface of the bed-rock, thus being continually exposed to capture by tree and brush roots which extend to bed-rock. If it is true

that most of the waters of deep percolation in mountain areas are to be found in open joints, it is apparent that cover conditions which serve to preserve the soil mantle have no effect on maintaining a storage reservoir for such waters. On the contrary, the soil mantle may serve to hinder rapid passage of the moisture into the open joints and, by intercepting moisture and retaining it for plant use, may decrease the total deep percolation.

Thus, assuming the soils to be the same, compare the effect of a deep mantle *versus* a shallow one. If the root zone extends to bed-rock in both cases, the root zone storage (field capacity) of a 10-ft soil mantle will be twice that of a 5-ft soil mantle, and it will require twice as much water to fill the 10-ft soil column if the moisture deficiencies of the two columns are proportional to their depths. In this case deep percolation would start sooner in the 5-ft soil mantle than in the 10-ft mantle. It will be found, in general, that the deep soil mantle will support a more luxuriant cover, due to the greater moisture storage ability of its deeper soil column. This greater storage ability for the benefit of wild growth causes a reduction in the residual quantity available for capture by Man.

Hydrological studies of valley and foothill sections in Southern California have led the writer to the following opinions:

(1) General deep percolation in these sections occurs only in years of exceptional rainfall. It occurs in spite of, rather than because of, the native cover.

(2) The native cover has adapted itself to its water supply and under usual conditions is able to convert nearly all moisture that penetrates the soil to its uses.

(3) If economically practical, an appreciable increment to ground-water storage can be made available by "debrushing" of waste lands on valley floors and outwash areas.

There remain to be explained certain water supplies that can not be accounted for by totaling the inflowing waters to closed basins. These additions to ground-water supply must result from the concentration of rain water in pools and streams and the resultant local deep percolation of rainfall as distinguished from general deep percolation.[57]

The observations on which the foregoing opinions are based, relate to valley and foothill sections of relatively small normal percipitation. Such sections are notably poor water yielders due to conditions of high evapotranspiration losses relative to total precipitation. Data for the higher, more productive, water-shed areas are lacking. It is indicated, however, that consumptive use increases with an increase in precipitation. The evaporation opportunity becomes quantitatively greater and native cover more dense as precipitation increases. The statement previously made with reference to valley and foothill areas is probably likewise applicable here, namely, that deep percolation occurs in spite of, rather than because of, native cover. Much information is needed before definite concepts can be formed.

[57] "Water Supply from Rainfall on Valley Floors", by A. L. Sonderegger, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1242; see, also, discussion of this paper, p. 1269 *et seq.*

The only data available at present for studies of such productive water-shed areas are such as those presented by the authors in their timely and valuable paper. Resultant data of this nature are inadequate, however, for intelligent planning of water-shed control. The factors entering into the final results, as reflected by stream-flow measurements, are too many and too conflicting in their influences to permit final conclusions to be drawn as to how a particular water-shed or part of a water-shed should be managed.

In the final analysis the problem must be approached by hydrologic rather than hydrographic methods. The latter are valuable in checking the former, but are not sufficient in themselves. Evaluations of the separate factors of evapo-transpiration, deep percolation, soil moisture storage, and local surface run-off must be made for typical water-shed areas with characteristic cover types, before it will be possible to arrive at useful conclusions. Of course, these factorial evaluations must be supplemented by other studies of silting, damage to property, etc., but these considerations, although necessary, are apart from the present discussion.

In connection with the mention of the necessity for separate evaluations of consumptive use by different types of cover, the writer would like to have the authors discuss the subject of relative losses from the predominant chaparral cover of the Southern California water-sheds as distinguished from the water-loving growth found only in stream bottoms. From the writer's studies in a similar water-shed the possibility is suggested that the major portion of the increased summer flow after burning may have to be credited to reduction in consumptive use along the stream bottom, and that widespread burning of the chaparral cover may have had only slight effect on the low-water flow of the stream.

In conclusion, the writer wishes to urge that engineers and foresters, charged with the task of attacking the perplexing problems of water-shed management, make a thorough study of soil moisture behavior and the concepts of hydrology with respect to consumptive use of cover before committing themselves to definite policies. The authors are to be congratulated on their valuable contribution to knowledge.

ROBERT E. KENNEDY,[68] M. AM. SOC. C. E. (by letter).[68a]—Another rumor run to earth, one less alibi for procrustean argument! Heretofore, when it became necessary or desirable to explain more or less run-off, evaporation, or percolation as the case may be, it could be ascribed to the fact that Man had cut down the trees, drained the land, plowed up the sod, or done something to the water-shed. This paper contributes materially toward "nailing down" the bogy of deforestation.

Devils Lake, in North Dakota, has become known extensively as a classic example of the assumption that the same conditions which result from deforestation have operated to reduce its elevation about 16 ft since the first immigration to the water-shed in the Eighties. In this case, it was not deforestation but the changing of the tight prairie sod to open porous wheat soil. The result is said to be the occurrence of less run-off into the lake by

[68] State Engr., Bismarck, N. Dak.

[68a] Received by the Secretary February 2, 1933.

reason of the increased loss due to more subsoil percolation, more plant consumption, and to crop exportation.

Much investment—Federal, State, and private—will be impaired if the lake finally dries up. One of the chief arguments for an enormous and fantastic engineering project to obtain water from the Missouri River, 100 miles to the west and 265 ft higher in elevation, was the assumption that the lake would not come back to any extent by natural causes, as other no-outlet lakes have done, because the introduction of the plow to the water-shed had upset the delicate balance in meteorological factors affecting it.

However, it seems evident that whatever other factors may prevail those conditions usually resulting from deforestation do not apply here because apparently they produce more run-off rather than less. Moreover, it will be difficult to prove that the mere changing of the soil cover from one type of sod to another has any appreciable effect.

The gauge heights do not show it. The lake fell 11.3 ft in the 20 years from 1867 to 1887. This is $6\frac{3}{4}$ in. per year. From 1887 to October, 1932, a period of 45 years, it fell 16.4 ft, or at the rate of $4\frac{1}{3}$ in. per year. During the first twenty years of the record, only climatic conditions could affect it. If human agencies have modified it since, the evidence must be sought elsewhere.

A readily computable factor frequently quoted is the quantity of water shipped out in farm products; but that is negligible. Eighteen inches of rain each year is the equivalent to 2 000 tons to the acre. Fifteen bushels of wheat to an acre contains only about 130 lb of water; but according to the report of the State Commissioner of Agriculture only about one-half the land was plowed in 1928 and 1929. This would mean that not more than 75 lb of water per acre existed in grain products and that not all of it was exported.

It might even be asserted with some display of statistics that somewhat more run-off rather than less could result from the introduction of cultivated crops. Be that as it may, there is probably little run-off at any time. The terrain is a pan and pot-hole, knob and kettle type of pleistocene glacial drift lying on cretaceous shale of the Pierre formation,[59] concerning which Professor Howard Simpson, of the University of North Dakota, states, "the region is almost unmarked by drainage lines save a few shallow winding coulees through which little or no water flows to-day."

There is a reported subsoil inflow of considerable but unmeasured quantity. If it has increased with the coming of the white man that development would tend to raise the lake rather than lower it; but there has been no report of a noticeable increase in the elevation of the ground-water table. No extensive land drainage projects have become necessary.

The lake has no doubt been dry or nearly so a number of times before. The same forces that filled it before are in all probability still operating without hindrance. It may be that, in the far distant future, great engineering works will be necessary to keep lake water out of the City of Devils Lake itself, now about 50 ft, or more, above the present water surface.

[59] *Bulletin No. 598*, U. S. Geological Survey, p. 73.

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DISCUSSIONS

TESTS FOR HYDRAULIC-FILL DAMS

Discussion

BY MESSRS. JEPTHA A. WADE, AND STANLEY M. DORE

JEPTHA A. WADE,[25] M. AM. SOC. C. E. (by letter).[25a]—The soil hydrometer described by Mr. Hatch is of interest where approximate results will serve and the elutriation method requiring more elaborate laboratory equipment is not justified. Apparently, judged by empirical standards, the results are consistent and close enough for ordinary purposes. By this method a good check can be kept on the materials entering a dam. It may also be used in determining whether the available dam material is suitable for hydraulic placement, or whether a rolled embankment is to be recommended due to excessively small grain size. It has been used, on work under the writer's charge, in analyzing samples from core borings in soft foundations to determine the probable seepage underneath an earth dam. Other applications will doubtless appear. Due to its comparative simplicity, both of apparatus and manipulation, it is undoubtedly a useful tool, and Mr. Hatch is to be commended for having developed the method for its use.

It is somewhat to be regretted that the paper does not go more fully into the underlying theory, with a frank statement of the factors that have been neglected and a determination or estimate of their limiting values. As it stands, the discussion is not entirely clear, considerable reliance being placed on the fact that the results agree fairly well with those obtained by elutriation.

The statement (under the heading, "The Hydrometer") of what is happening in the sample under test, is somewhat confusing—in fact, not strictly true. Assuming that the graduate contains a uniform mixture from top to bottom and that settlement takes place in accordance with Stokes' law, there will be particles of all sizes at all levels when the test is started. All these particles immediately begin to settle at rates dependent on their respective

NOTE.—The paper by Harry H. Hatch, M. Am. Soc. C. E., was presented at the Joint Meeting of the Irrigation and Power Divisions, Yellowstone National Park, Wyoming, July 7, 1932, and published in October, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1933, by Charles H. Paul, M. Am. Soc. C. E., and February, 1933, by Joel D. Justin, M. Am. Soc. C. E.

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[25a] Received by the Secretary January 13, 1933.

diameters. Each horizontal section will gain from above the same number of particles of a given diameter as it loses to the section beneath, the only exception being at the top where none can be supplied and the number of particles in suspension, therefore, becomes smaller. Particles originally near the bottom begin to come to rest on the bottom of the graduate. As time goes on, the available supply of particles of a given size above a given section becomes exhausted and the total grams in suspension per liter at that point is decreased. There are still particles of all the smaller sizes above this point, and of all sizes below this point.

Conditions at the various levels at any time can be determined if the analysis of the material in suspension is known. Assume that this is as given in Table 3 of the paper, or that the graduate contains:

45.58 grams smaller than 0.09 mm in diameter (200-mesh screen)
27.2 grams smaller than 0.0467 mm in diameter
19.7 grams smaller than 0.0345 mm in diameter
11.9 grams smaller than 0.0230 mm in diameter
8.9 grams smaller than 0.0165 mm in diameter
5.9 grams smaller than 0.0097 mm in diameter
4.2 grams smaller than 0.0069 mm in diameter

Suppose these quantities are plotted as abscissas marking the vertical lines between the bands with the separation sizes. Next, apply Stokes' law, determining the distance a particle of each separation size would drop in each of the times for which readings were taken. Plot these distances as ordinates measured down from the top, which represents the surface of the liquid. In 1 min each particle, 0.09 mm in diameter, will have dropped 44.58 cm. There will be no particles as large as this nearer the surface, but at greater depths the number of this size per liter will be uniform and the same as at the beginning. Particles, 0.0467 mm in diameter, will all have traveled 11.94 cm, those 0.0345 mm in diameter, 6.54 cm, etc. If these distances are plotted on the lines representing these sizes and a curve is drawn through the points, the result is a curve above which no particle of the size represented by the abscissa exists at the end of 1 min and below which conditions are the same as they were at the beginning. Since horizontal distances represent grams per liter the abscissas of the curve represent the total grams per liter at the various depths below the surface at the end of 1 min. Similar curves can be drawn for the other time intervals as shown in Fig. 20.

The hydrometer depends on the density of the displaced liquid for its operation. While there may be a question as to suspended solids increasing the density of the liquid it appears that, within certain limits at least, water with material in suspension affects the hydrometer generally as if this were true. While this suggests itself as a subject that might properly be investigated, no such attempt will be made here, the assumption being that the relation is a true one. It now appears that, in the work under discussion, the hydrometer is being used in a very peculiar manner, namely, in a liquid having different densities at different depths, increasing from the top down. What is the resultant effect? The buoyancy exerted on any floating body is

equal to the weight of the displaced liquid which must also equal the weight of the floating body. The hydrometer reading indicates a summation of all the buoyant forces acting on the hydrometer; that is, each horizontal section of displaced volume multiplied by its respective density. The reading might be termed the resultant of all these forces.

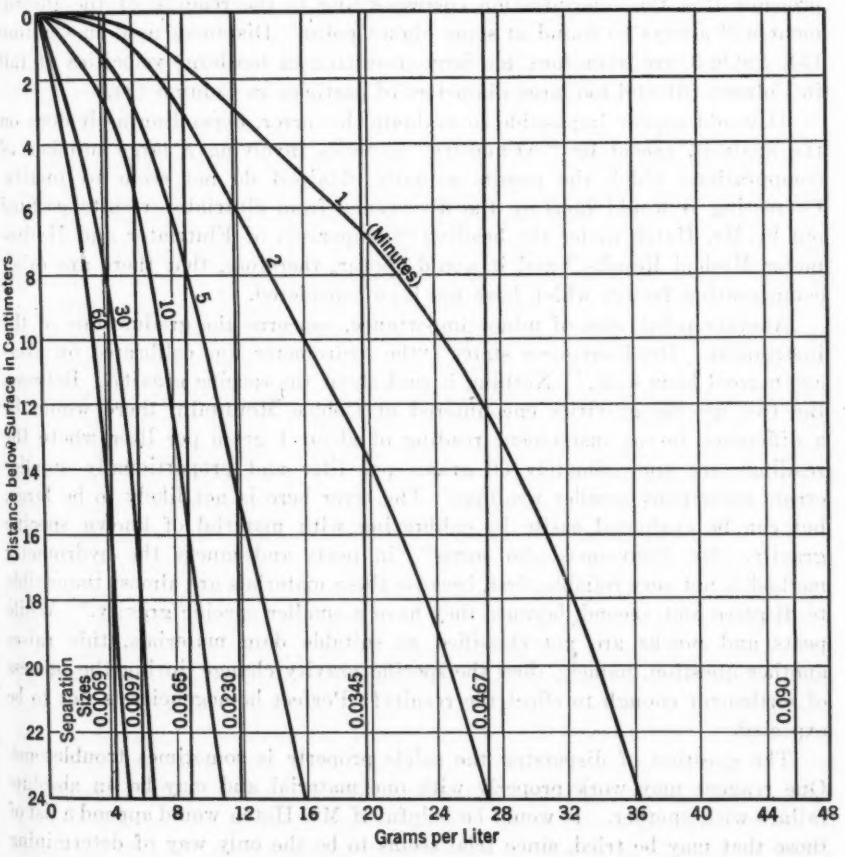

FIG. 20.—GRAMS PER LITER AT DIFFERENT DEPTHS AT TIMES READINGS ARE TAKEN. (ANALYSIS OF MATERIAL AS IN TABLE 3.)

The use of the hydrometer for analysis purposes, therefore, depends on finding the horizontal plane in which the actual grams per liter in suspension, if it was constant throughout the displaced volume, would produce the same effect as is actually obtained. Mr. Hatch assumes that this level is at the center of buoyancy of the hydrometer, but examination of Fig. 20 indicates a slight error in the assumption. Suppose, for the moment, that the hydrometer was cylindrical in form. The center of buoyancy would then be exactly half way between the surface of the liquid and the bottom of the hydrometer.

The density lines will usually be convex upward, as they are in Fig. 20, since any other hypothesis denotes an extraordinary preponderance of the larger sizes. The density at the mid-point will obviously be greater than the mean, and the density equal to the mean will be found at a higher level. Since the center of buoyancy is the center of gravity of the displaced liquid, it is probable that the concentration corresponding to the reading of the instrument will always be found at some higher point. Distances used in Column (5), Table 3, are, therefore, too large, resulting in too large velocities of fall in Column (6) and too large diameters of particles in Column (8).

It would appear impossible to evaluate this error, depending as it does on the analysis, except by "cut-and-try" methods involving a large number of computations which the results actually obtained do not seem to justify. Correcting it would increase the divergence from elutriator results pointed out by Mr. Hatch under the heading "Comparison of Elutriator and Hydrometer Method Results," and it would appear, therefore, that there are other compensating factors which have not been considered.

Another point, also of minor importance, concerns the graduations of the instrument. Dr. Bouyoucos states[10] "the hydrometer was calibrated on average normal loam soils." Nothing is said about the specific gravity. Between the two specific gravities encountered at Cobble Mountain, there would be a difference in the instrument reading of about 1 gram per liter where the readings are approximately 60 grams per liter and proportionally smaller errors accompany smaller readings. The error here is not likely to be large, but can be evaluated easily by calibrating with material of known specific gravity. Dr. Bouyoucos also states[10] "in peats and mucks the hydrometer method is not very reliable, first, because these materials are almost impossible to disperse and, second, because they have a smaller specific gravity." While peats and mucks are not classified as suitable dam materials, this raises another question, namely, does the specific gravity change during the process of settlement enough to effect the results? Perfect homogeneity is not to be expected.

The question of dispersing the solids properly is sometimes troublesome. One reagent may work properly with one material and may be an absolute failure with another. It would be helpful if Mr. Hatch would append a list of those that may be tried, since trial seems to be the only way of determining this vital point.

It is not felt that any of the points discussed are of enough importance to put the method in serious doubt. Rather, it is felt that discussion of them is essential to a better understanding which must precede the general adoption of the method and perhaps lead to its improvement. It is to be noted that temperature corrections have been applied in Column (4), Table 3, that are much closer than it is possible to read the hydrometer and that the viscosity is 1.00 for a temperature of 68° F. It would seem possible to control the

[10] "A Comparison of the Hydrometer Method and the Pipette Method for Making Mechanical Analysis of Soils with New Directions," by George John Bouyoucos, *Journal*, Am. Soc. of Agronomy, August, 1930, Vol. 23, No. 4, p. 750.

temperature closely enough so that these corrections might be neglected, without causing important errors. This would simplify operation considerably. It is quite probable that a few check runs with an elutriator on the material from a given location will determine a fairly constant correction by which hydrometer and elutriator results may be brought into close agreement. The errors, with the exception of temperature, evidently have some relation to the analysis. Mr. Hatch very properly points out that the individual hydrometer must be calibrated, and it is believed that, if this is done and an occasional check made in a laboratory equipped for elutriation, the hydrometer may be used with entire confidence.

STANLEY M. DORE,[26] ASSOC. M. AM. SOC. C. E. (by letter).[26a]—Engineering supervision of the construction of large dams of the earth embankment type has included first, the inspection and choice of construction materials, and, second, the regulation of the manner of incorporating the materials chosen into the structures. In recent years the soil testing laboratory has taken an important and active part in the study of materials in connection with both these phases of the supervision, and the decisions made before and during construction have been influenced considerably, and justly so, by its findings. Construction of dams, by the hydraulic or the semi-hydraulic methods especially, has been dependent upon and governed by the periodic analyses and tests of the laboratory, in order that the quality of the material in the finished structure may be ascertained. Thus, the findings of the laboratory for any such project constitute data of vital significance and utmost value to those interested in earth embankment design and construction.

The manner in which testing has been conducted on recent large projects has varied considerably, however, the engineers on each devising and inventing their own methods and apparatus to a large extent and interpretating their own data. The result has been that the findings obtained for one project have not been comparable with those for another, and for that reason many significant data have failed to be as valuable as they should to the Engineering Profession. The need for standardization of methods and apparatus is decidedly apparent. The author's appeal for such standardization is to be considered seriously and energetic action, through engineering organizations, toward that end is strongly recommended. Much has been done under the direction of such governmental departments as the U. S. Bureau of Public Roads, the U. S. Bureau of Agricultural Research, and the U. S. Geological Survey, and papers and reports dealing with the subject have been written by Messrs. Hazen, Slichter, King, Stearns, Meinzer, Terzaghi, and others. Many of these works may be of use in the standardization of soil testing, but considerable thought and discussion of the problem are urgently needed at present.

Soils, the most abundant natural construction material, have quite variable structural characteristics—even similar soils in one locality. Adequate standard terms for discussing these variable characteristics are lacking to some

[26] Asst. Designing Engr., Met. Dist. Water Supply Comm., Boston, Mass.

[26a] Received by the Secretary January 16, 1933.

extent and definition of them in simple practicable terms might well constitute the first step toward standardization. Even though two materials to be compared have radically different characteristics, the existence of common definitions would permit the description of these differences so clearly that studies of the results of analyses made of them would be productive of useful data. In contrast, the lack of common adequate phraseology and definitions makes comparisons of test results futile as the true differences of the materials compared are not fully understood, simply because at present this deficiency hampers minute and clear descriptions of those differences.

The mechanical analysis of a soil sample is probably the most widely used single test of soil characteristics, and attention is directed to the author's recommendation for the careful rating of sieves and for the accurate determination of the finer particles. For the latter he indicates that successful results can be obtained with either the elutriation or hydrometer method, and that the use of both may be advisable. In addition, it seems that considerable might be done in recording other features in connection with this analysis, such as the determination of the shapes of the grains, the ratio of the percentages of each different type of shape to the whole, the surface characteristics of the grains (whether sharp, smooth, angular, ragged, rough, etc.), and the chemical, physical, and geological make-up of all the several grains, and, where possible, their specific gravities.

Microscopic examination and measurement should be used more extensively. Too little attention is now given to these features, which greatly affect the structural qualities of the soil. When the mechanical analysis curve of a given sample is recorded graphically, as is commonly the case, the results of microscopic and other examinations and measurements might well be included on the same sheet. Then, for example, when the permeable qualities of this sample are compared with those of another similarly tested, more competent and reasonable conclusions could be reached and relations between the permeable qualities and the mechanical analyses and grain shapes might be determined.

At present, a comparison of mechanical analyses curves really gives little information of the relative structural characteristics of two samples, but only the differences and likenesses in respect to distribution of sizes of the grains. The sizes determined by sieving are not even strictly comparable as the sieves do not always separate grains from one sample of the same volumetric size as those from another, because of the effect of the shape and surface properties of the grains upon the sizes separated during sieving. This condition is true also to some extent in the measurement of finer grains by elutriation or hydrometer methods. Briefly, it is believed that the shape of the grain has fully as much influence on the structural properties of a soil as the size, and to the present time laboratory methods in general offer no adequate means of analysis and study of the effects of the shapes of the soil grains.

"Soil engineering" has advanced with rapid strides in the past decade and probably will progress at an accelerated rate in the next ten years, so it seems that the time is "ripe" for consideration of standardization of testing methods.

The apparatus used in the Cobble Mountain Testing Laboratory was given considerable thought as to its make-up, and careful attention as to its performance during use. The methods of determining grain-size analyses by sieving and by hydrometer or by elutriation have similarly been used elsewhere, and as these methods and the results obtained are well described and analyzed in this paper, they might well be copied in detail for use on other projects. The tests for turbidity, moisture content, and organic content are, at present, in common use elsewhere. The apparatus described for the measurement of the coefficient of friction, although not generally used, seems worthy of consideration for adaptation for future testing as it seems to have given the required results. More definite knowledge, however, is desirable regarding the effect of moisture content on tests made with this machine as well as the effect of crushing of grains during testing.

The tests for percolation, or permeability, and for porosity, however, seem to be the most difficult for engineers to agree as to which method of testing gives the most satisfactory results. Although the author did not state which of the three methods for testing permeability (the large percolator, the standpipe permeameter, and the standard compression apparatus) used at Cobble Mountain he considered the best, he did infer that any of these three would give reliable results and that results from the horizontal and vertical capillary tube methods were unsuitable.

Without attempting to discuss the pros and cons of any of these methods, the results obtained from any two of them are not comparable, as each method introduces individual features of testing. For instance, the capillary tubes are usually packed dry; many of the can type of tests are packed tacky or moist; and, for some soils, comparisons have shown that the former will give much larger percolations. Apparently, when the samples are dried there is a breaking apart of particles or grains held closely together, and the permeability is increased thereby. Tamping the samples has a marked effect on the permeable result obtained, and the moisture content during tamping increases this effect.

To obtain field and laboratory conditions of testing that will give results of the material in its natural state and in its final position in the structure, considerable judgment must be exercised in making representative tests. Care is also needed to describe the test conditions clearly in explaining the results obtained. Mixing the sample seems to decrease its permeability. The mixing seems to distribute the fines throughout the mass more uniformly in position so that the permeability of the samples is less than in its natural state. Handling the materials to be tested influences the results in such a manner that it is practically useless to discuss various apparatus until a common method of taking and handling the samples can be adopted.

However, the type of apparatus to be used must be given consideration. Large and small cans, permeameters varying in size, large tank percolators, and vertical and horizontal capillary tubes have been used with varying degrees of success. The results obtained are not comparable because each individual type has inherent characteristics that affect the results in a sig-

nificant manner. It is of prime importance that an acceptable standard type of apparatus, even though not considered perfect, be generally adopted and used to aid in comparative studies of soil permeabilities.

The term, "effective size," as used by the author, is that of Hazen, and a relation between the 10% size of the core material and permeability was determined. In perusing his results, care should be taken not to confuse this term with that of Slichter because they are not alike by definition. Furthermore, the constant factor of 150 should not be used on other work because, when found for any other materials, this factor may vary as much as several hundred per cent. from that figure. In fact, it is believed that during the testing, the author must have found wide variations from the mean used, since it is extremely difficult to establish any relation between the 10% size and permeability that will hold for a range of samples, especially with varying uniformity coefficients. The use of any formula for permeability containing the term, "effective size" (meaning the 10% size), is limited, and the results obtained by its use should be regarded only as rough estimates.

Some surprisingly simple formulas for consolidation of the core and seepage are developed in the paper. Assumptions are made, however, which affect their sureness and accuracy, although they should prove useful as guides to other estimators. Drainage through the core is probably mostly in a diagonal line, while the formulas developed are based on the assumption of horizontal flow of water. The introduction of the arbitrary "effective size" gives a term in the formulas that is likely to upset their accuracy when used elsewhere, unless considerable testing is done to determine the proper coefficients.

In general, this paper, in describing testing methods, apparatus, formulas used, and results derived, offers a particularly valuable contribution to the literature available on soil testing, and this branch of soil engineering offers a large and attractive field for engineering research and study now and in the near future.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FOUNDATION TREATMENT AT RODRIGUEZ DAM

Discussion

BY MESSRS. L. A. ROBB, FRANCISCO GOMEZ-PEREZ, AND F. A. NOETZLI

L. A. ROBB,[3] M. AM. SOC. C. E. (by letter).[3a]—The foundation arch, the side-hill bracing walls, and the deep cellular-wedge type of cut-off wall of the structure described by Mr. Williams in his timely and interesting paper are innovations in dam design that may have a far-reaching effect. Faulted foundations and broken-rock banks, such as have been discussed by the author, will be encountered more frequently in the future because sites consisting entirely of sound rock are becoming increasingly scarce; and yet the development of these faulted and otherwise unsatisfactory sites are frequently of great economic importance.

Certain principles of foundation design and construction as presented by the author, with modifications, may be applied broadly to other difficult foundation problems of a similar nature. Aside from foundations which uniformly have low bearing value, such as clay or sand, difficulties are more frequently being encountered with: First, those foundations in which there exist the danger of horizontal or inclined movement, owing to seams and planes of cleavage; and, second, foundations consisting of materials varying widely in bearing value.

In the first type, movement may occur from earth or rock slides in the bank, due to planes of cleavage in the foundation material, or from more or less horizontal planes of cleavage in the rock below the dam foundations. Such movements caused the failures of large gravity dams at both Austin, Pa., and Austin, Tex.[4]

NOTE.—The paper by Charles P. Williams, M. Am. Soc. C. E., was presented at the meeting of the Irrigation Division, Sacramento, Calif., on April 24, 1930, and published in October, 1932, *Proceedings*. This discussion is published in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[3] Vice-Pres., Amburgen Construction Co., New York, N. Y.

[3a] Received by the Secretary January 10, 1933.

[4] *Engineering News-Record*, October 19, 1911.

Foundations of this type require that the seamed rock banks be braced either across the valley or anchored firmly to sound rock below in such a manner that the seams are water-tight and that lateral or longitudinal movements will be impossible. At the Rodriguez Dam the method of bracing the seamed rocks across the valley was used, as shown by Fig. 7.

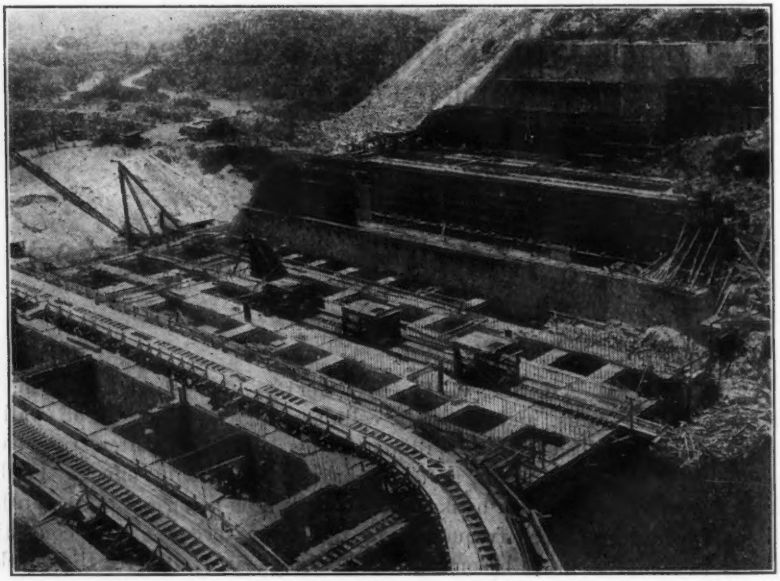

FIG. 7.—VIEW SOUTHWEST FROM NO. 2 CONCRETE PLANT.

Similar shattered bank conditions were encountered at another site recently studied. However, in this case, the valley was too wide to use bracing ribs. In addition, this problem was complicated by the fact that the side hills were of such height that loosened or broken material would have made excavation extremely hazardous. A foundation structure, involving a modification of the cellular-wedge cut-off construction described by the author, was worked out as a possible solution for this problem.

Fig. 8 shows the suggested design. It was proposed to construct four lines of the cellular-wedge type of cut-off walls, starting in the river bed and working upwardly into the bank. It was also proposed to carry the foundations of these walls well into sound rock and to converge the four lines as shown by Fig. 8 so that the up-stream and down-stream walls would coincide with the heel and toe of the dam. The footings are designed as an integral part of these walls creating in effect a large subterranean reinforced concrete frame which would be capable at conservative stresses of carrying any possible thrust from the shattered or seamed side hill to the sound rock in the river bed below, and, in addition, would firmly anchor the dam to a sound foundation throughout its entire structure.

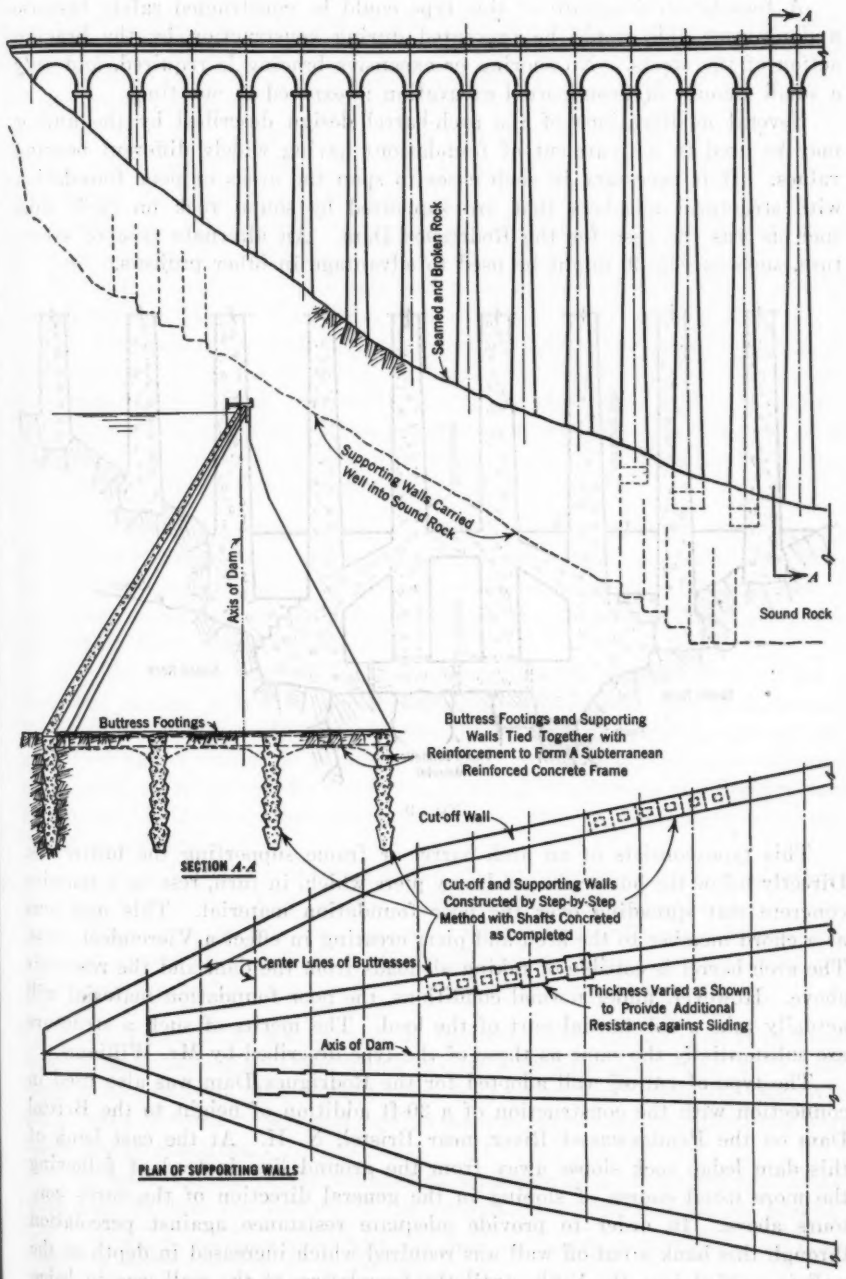
A
Seamed and Broken Rock
Supporting Walls Carried Well into Sound Rock
A
Sound Rock
Axis of Dam
Buttress Footings
Buttress Footings and Supporting Walls Tied Together with Reinforcement to Form A Subterranean Reinforced Concrete Frame
Cut-off Wall
SECTION A-A
Cut-off and Supporting Walls Constructed by Step-by-Step Method with Shafts Concreted as Completed
Center Lines of Buttresses
Thickness Varied as Shown to Proivde Additional Resistance against Sliding
Axis of Dam
PLAN OF SUPPORTING WALLS

FIG. 8.

A foundation structure of this type could be constructed safely because a dangerous slide would be prevented during construction by the bracing action of the struts. No shoring or expensive bracing is required, and only a small amount of unsupported excavation is exposed at one time.

Several modifications of the arch-barrel design described by the author may be used in a treatment of foundations having widely different bearing values. It is necessary in such cases to span the areas of poor foundation with structural members that are supported by sound rock on each side, such as was the case for the Rodriguez Dam. An alternate type of structure, such as Fig. 9, might be used to advantage in other projects.

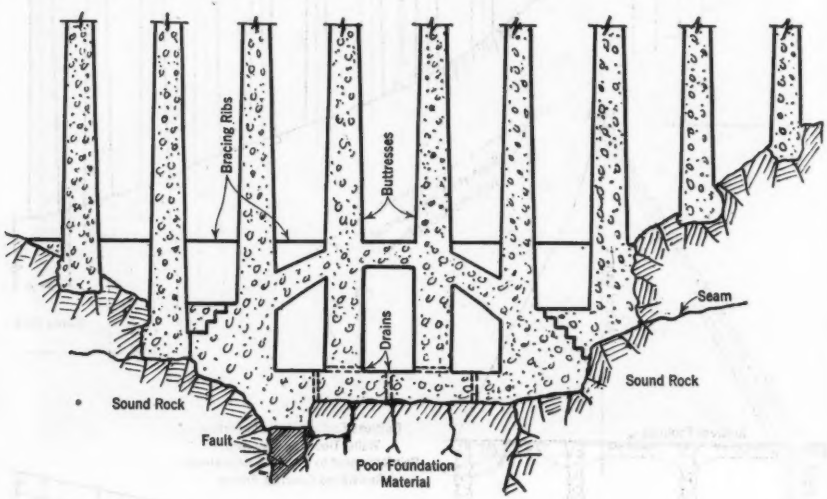

FIG. 9.

This type consists of an arch barrel or frame supporting the buttresses. Directly below the buttresses are heavy piers which, in turn, rest on a massive concrete mat spreading over the poor foundation material. This mat acts as a chord member to the arch and piers creating in effect a Vierendeel truss. The arch barrel is capable of taking all loads from the dam and the reservoir above. However, under normal conditions, the poor foundation material will actually take a substantial part of the load. The merits of such a structure are substantially the same as those of the type described by Mr. Williams.

The type of cut-off wall adopted for the Rodriguez Dam was also used in connection with the construction of a 30-ft addition of height to the Bristol Dam on the Pemigewasset River, near Bristol, N. H. At the east bank of this dam ledge rock slopes away from the ground line instead of following the more usual course of sloping in the general direction of the earth contours above. In order to provide adequate resistance against percolation through this bank a cut-off wall was required which increased in depth as the wall is carried into the bank, until the foundation of the wall was in ledge rock about 110 ft below the ground surface.

The concrete cut-off for the Bristol Dam was carried down through clay, gravel, and boulders, at the rate of about 1 ft in depth per day. Provisions were made to grout the joints between the sections of wall. The grout pipes also served as drains for the purpose of relieving hydrostatic pressure during construction. The advantages of using this type of wall are, as follows:

1.—The necessity of excavating a wide, deep trench and of using expensive and dangerous shoring is avoided.

2.—Wedge action firmly compacts the material supporting the wall and thus greatly increases the resistance of the wall to water travel.

3.—This procedure offers a safe and rapid method of constructing a deep cut-off wall and involves a minimum amount of exposed excavation.

4.—The wall is not endangered by flood hazards while it is in process of construction. If in the river bed, during the flood season, the cellular spaces may easily be protected against filling with silt and débris and only a minimum amount of pumping will be required to empty the cells after the flood has passed, or the shafts may be carried up above flood-water level and construction continued uninterruptedly.

FRANCISCO GOMEZ-PEREZ,[5] JUN. AM. SOC. C. E. (by letter).[5a]—In his paper the author has covered his subject in so thorough and complete a manner that it is difficult to add anything without having to enter into other phases of the construction which are independent of the foundation and, therefore, not within the scope of the discussion.

A point of view implied in the paper but which, because it is not of a technical nature, is not developed, brings attention to the fact that even in the face of difficulties of various kinds, Mexico is doing what she can to proceed with the construction of public works. These include some of National scope, such as irrigation developments, in which members of the Society have co-operated. The technical aspects of some of these projects have been published by the Society.

It is to be hoped that the association of engineers from the United States, endowed with such a high technical and cultural background, and such personal integrity, with Mexican engineers, will tend toward mutual understanding and appreciation, which, in time, will bring about a more cordial and effective co-operation.

F. A. NOETZLI,[6] M. AM. SOC. C. E. (by letter).[6a]—The unusual foundation conditions at Rodriguez Dam, which involve a geologic fault about 20 ft wide, presented a problem that could be met only by unusual methods. Mr. Williams has presented a clear and concise description of how the difficulties were successfully overcome.

The main feature of the foundation structure is a horizontal foundation arch that bridges the bottom of the canyon with a clear span of, roughly,

[5] Res. Engr., Rodriguez Dam, with the Govt. of the Northern Territory of Lower California, Mexico, Tijuana, B. C., Mexico.

[5a] Received by the Secretary January 25, 1933.

[6] Cons. Hydr. Engr., Los Angeles, Calif.

[6a] Received by the Secretary February 7, 1933.

70 ft and extends for a distance of approximately 250 ft in an up-stream and down-stream direction. This long arch barrel supports four of the highest buttresses of the dam.

At first, the problem of the design of a foundation arch for supporting buttresses nearly 250 ft long, seemed to present almost insurmountable difficulties from a theoretical point of view, mainly because of the necessity of taking into account the arch deflections resulting from the heavy concentrated loads exerted by the buttresses.

The foundation arch had to meet the following requirements: (*a*) The axis of the arch barrel was to be parallel to the buttresses so as to avoid undesirable twisting moments; (*b*) the span, radius, and thickness of the elementary arches of the arch barrel were to be of such a size that the theoretical arch deflections along the base of each buttress would be as nearly as practicable on a straight line and would correspond to the trapezoidal stress at the base of the buttresses.

Fortunately, the bed-rock conditions along both side-hills at the bottom of the canyon made it possible to meet these requirements very closely. For some parts of the arch barrel it was necessary to use arch spans of greater length than would have been necessary to span the fault and the rock of inferior quality, and, for certain other parts, some extra excavation was required; but, in general, the conditions may be said to have been rather favorable for a foundation structure of the type adopted.

The structural analysis of the individual elementary arches was made by the elastic theory and offered no unusual difficulties. For the purpose of design the arch barrel was divided into strips from 10 to 20 ft long. The radius, span, and central angle, and the thickness of each strip at crown and abutment were then determined by "cut and trial" such that the necessary conditions regarding deflections and permissible maximum unit stresses were met within reasonably close limits. It is of importance to note that the buttress walls that transmit the load to the foundation arch are relatively stiff in a vertical direction. They will tend, therefore, to force a practically linear deflection of the foundation barrel to occur along the base of each buttress, regardless of the results of the theoretical analysis. For small differences between theoretical and actual straight-line deflections the loads on the individual strips will depend upon their relative stiffness, such that the stiffer parts will carry somewhat more than the theoretical load, but will thereby relieve the relatively more flexible parts by the same amount of load. For instance, if one strip of the arch barrel were overloaded by, say, 5% of its theoretical load, the adjoining strip or strips would be relieved by approximately this same amount so that under actual load conditions the deflections of the arch barrel would again vary practically linearly along the base of each buttress. For safety, the lower part of each buttress is strongly reinforced, as shown in the author's Fig. 3.

While the foundation arch is designed to carry the full superimposed load, no doubt a considerable part of this load will be supported through the sub-intradosal concrete by the natural foundation material in the bottom of the canyon.

Small irregularities in settlements of one buttress relative to the others are not likely to affect the stability of the dam, because of the relative flexibility of this type of structure.

One interesting feature of the foundation structure of the Rodriguez Dam is the provision for admitting water through inlet pipes along the intrados of the foundation arch, whereby artificial uplift pressure is created which, in periods of floods, counteracts the heavy load on the foundation arch, due to back-water entering between the buttresses.

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DISCUSSIONS

A HISTORY OF THE DEVELOPMENT OF WOODEN BRIDGES

Discussion

BY MESSRS. C. J. HOGUE, WILLIAM A. OLIVER, PHILIP G. LAURSON, RICHARD S. KIRBY, WELLS N. THOMPSON, BENJAMIN WILDER GUPPY, IVAN E. HOUK, AND WILLIAM H. BAKER

C. J. HOGUE,[46] M. AM. SOC. C. E. (by letter).[46a]—This paper is one of great historical and informative interest and value, at least to one connected with the timber industry, and the writer hopes that it may be to others, as well.

It brings the history and practice in wood bridges to date, and the writer believes that the present is the eve of a fourth era in wood bridge design and construction, which will be of even greater interest and importance, from both engineering and architectural viewpoints, than the three recorded in the paper.

Temporarily, bridge design and construction in wood have been somewhat eclipsed by other materials. These materials have been new; wood construction has been old. They have been replete with new technical studies and developments; wood technology, old as it is, has lacked definite information on some points.

To-day, as not for many years, the bridge designer is faced with the necessity for the utmost economy and, probably as never before, with the question of the length of useful life and early obsolescence. A timber supply is amply available, recent research has established authoritative information on most points in question, new methods of framing have been developed, and treated timber offers material of economic permanence, ease of remodeling, low cost of removal, and salvage value on removal.

During recent years, the Forest Products Laboratory, of the U. S. Forest Service, at Madison, Wis., has published the results of technical studies

NOTE.—The paper by Robert Fletcher and J. P. Snow, Members, Am. Soc. C. E., was published in November, 1932, *Proceedings*. Discussion of this paper has appeared in *Proceedings*, as follows: January, 1933, by Henry B. Seaman, M. Am. Soc. C. E.; February, 1933, by Messrs. Jasper O. Draffirm, E. K. Morse, and John W. Storrs.

[46] In Chg. of Technical Service, West Coast Lumbermen's Assoc., Seattle, Wash.

[46a] Received by the Secretary January 3, 1933.

which have taken the mystery and variance from the calculation of the bearing strength of wood posts, of bearing on wood at various angles to the grain, of the bearing strength of bolts on wood, of the calculation of shear in wood beams and stringers, and of the basic principles of calculating the strength of laminated wood beams and posts. The ring dowel connector, developed in Germany during recent years, opens a vast new field of economy and flexibility in wood framing.

It has been shown by experience that the durable life of treated timber may be assumed safely at forty years or more; it seems to be the consensus of opinion of bridge engineers that the useful life of a bridge in these times of rapid change may not be assumed safely at more than twenty to twenty-five years, often even less, due to early change in location.

The public has been taught to expect beauty as well as utility in its bridge structures. During 1932 a number of bridges of appealing architectural character, with little increase in cost, have been built in Oregon.[47] Such framing connectors as the ring dowel open the field for wood bridge construction to long-span arches and arched trusses of imposing size and appearance. The writer believes that a new chapter in wood bridge construction is about to be written.

WILLIAM A. OLIVER,[48] ASSOC. M. AM. SOC. C. E. (by letter).[48a]—The reading of this paper has been a pleasure and an inspiration. The education of an engineer is not complete until he has become acquainted with the work of the giants who have gone before him. It is through such efforts as the one under discussion that this part of an engineering training is obtained.

In their "Synopsis," the authors write, "but it will appear that, where timber is plentiful * * * wooden bridges still have 'a reason to be.'" Later, they state (see "Stringer Bridges"), "although it can be said that the day of wooden bridges has passed, it is a fact that many hundreds of miles of wooden trestles, stringer, and truss bridges are still in service in the United States." It is not entirely clear just what the authors' conclusion is regarding the present adaptability and economy of timber bridges. The writer would like to discuss the paper briefly from this point of view.

One of the important objections to timber bridges and structures generally has been the so-called "lack of permanency." The writer does not believe that the facts justify this idea. By proper maintenance a timber bridge may be just as permanent as a bridge built of some other material. For centuries, and by consistent and precise maintenance, the Japanese have made their timber bridges perpetual. Numerous examples of timber bridges several decades old can be found in the United States, and many of these bridges have had only ordinary care during these years of constant use. Certainly fifty years is a long life for any ordinary bridge of any material in the United States. Some examples of present-day wooden bridges are briefly discussed in the following paragraphs.

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[47] *Civil Engineering*, September, 1932, Vol. 2, No. 9, p. 549.

[48] Urbana, Ill.

[48a] Received by the Secretary January 3, 1933.

According to Mr. E. Jervoise,[49] Selby Bridge over the River Ouse, in Yorkshire, is probably the oldest timber bridge in England. It was built in 1790 and is still in use (1933). The bridge has a swing section which, in Fig. 31, is shown thrown from its pivot. Mr. Jervoise states that through this mishap the bridge was out of use for some time.

Of course, modern examples of wooden bridges are plentiful on the West Coast of the United States, but, in 1932, the State of Ohio found it expedient to build ninety-three timber highway bridges. Most of these structures were of a simple trestle type, built of timber produced in the State. All material

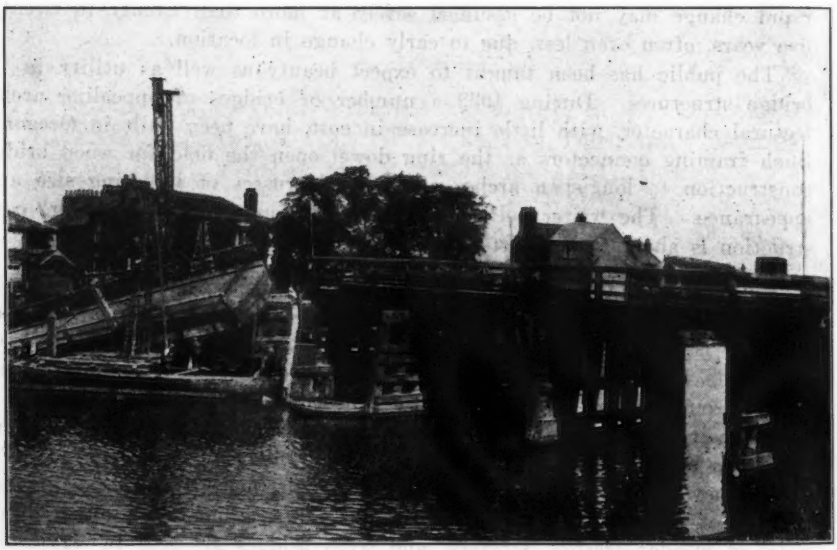

FIG. 31.—SELBY BRIDGE OVER THE RIVER OUSE IN YORKSHIRE, ENGLAND.

was impregnated with preservative, all framing being done in advance of treatment. It is interesting to know that a mid-Western State had timber in this quantity available for use, and that one of the more progressive States did find it economical to adopt such an elaborate program of timber bridge construction.

One of the earliest of materials of construction, timber has been developed for structural purposes by "rule-of-thumb" methods. Due to the inertia that these "rule-of-thumb" methods obtained through the years, it has been difficult to make the ordinary timber user appreciate the fact that greater economy as well as strength could be obtained through a more scientific use of the material. However, through the efforts of the various lumber manufacturers' associations and the work of the materials testing laboratories, this difficulty is gradually being overcome and the general trade is evincing more and more interest in researches in the use of timber. Many examples of

[49] Ancient Bridges of England Series, Architectural Press, Westminster, S. W., London, England.

present-day research could be given as examples of what is being done along these lines. As far as the testing laboratory is concerned timber is almost an ideal material, conforming beautifully with the accepted theories of mechanics.

Referring to the work of Messrs. Brangwen and Sparrow,[14] the authors state in their discussion of Roman bridges, that very little is known of the construction of Pons Sublicius, "hallowed by traditions." A longitudinal elevation and a cross-section of this bridge has been published[50] by Mr. H. G. Tyrell. The writer wonders whether the authors question the authenticity of this illustration.

PHILIP G. LAURSON,[51] M. AM. SOC. C. E. (by letter).[51a]—The authors have contributed a paper of great value to civil engineers who are interested in bridges. Much of the information is not available elsewhere.

It is natural and proper to emphasize the American developments as the authors have done. It is true, however, that wooden bridges were used in Europe a century ago to a greater extent than is commonly realized.

The *Minutes of Proceedings* of the Institution of Civil Engineers of Great Britain, and other European engineering publications, contain numerous descriptions of wooden railway and highway bridges. In 1846, John Rennie, in his Presidential Address, reviewed the developments in all branches of Civil Engineering, including wooden bridges. He reported at that time[52] that the trussed system had been applied with considerable success in some well-constructed bridges across the Tyne River and in several other places. The system of combining small curved pieces of timber connected in the form of an arch, adapted for large spans, was first introduced by C. F. Wiebeking in 1826, according to Rennie, when a bridge of 100-ft span was constructed on the Ancholme with complete success. He reported also that the lattice bridge had recently been introduced on the Birmingham and Gloucester Railway and on the Dublin and Drogheda Railway and that it had been found economical and simple in construction. A number of European types of wooden bridges, with details, was described by Benjamin Green in the same year[53] in a paper entitled "Arched Timber Viaducts."

Isambard Kingdom Brunel designed many timber railway bridges in a great variety of types.[54] Brunel was particularly concerned with the prevention of decay and, as early as 1835, was in communication with Faraday as to the best method of testing the extent to which Kyanizing solution penetrated into wood. It is reported that Brunel made a careful study of all the different methods of preserving timber, and used the more successful of them on a very considerable scale.

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[14] *Proceedings*, Am. Soc. C. E., November, 1932, p. 1461.

[50] "History of Bridge Engineering," p. 23.

[51] Assoc. Prof. of Eng. Mechanics, Sheffield Scientific School, Yale Univ., New Haven, Conn.

[51a] Received by the Secretary January 13, 1933.

[52] *Minutes of Proceedings*, Inst. C. E., Vol. V (1846), p. 33.

[53] *Loc cit.*, p. 219.

[54] "Life of I. K. Brunel, Civil Engineer," by Brunel, Lond., 1870.

Many of the European engineers of these early times were well-educated men of high technical skill. Carl Friedrich von Wiebeking (1762-1842), mentioned by Rennie, was Director-General of Roads and Bridges in Bavaria, and was the author of several treatises on hydraulics, mechanics, and structures. He worked out proportions for his laminated wood arches for spans as great as 600 ft. Bridges of this type were built in England and on the Continent in considerable numbers. The authors' Fig. 6 is of this type, but in most of the bridges the bent planks were thinner — 3 in. being a common thickness. As many as sixteen or twenty thicknesses of bent planks were bolted together to form the arch rib.

Another good source of information concerning European timber bridges is the set of publications on the "Theory, Practice, and Architecture of Bridges," edited and published by John Weale about 1843; it contains two volumes of fine plates. The early volumes of *L'Illustration* also contain pictures of wooden bridges in the days when railroads were still objects of curiosity.

It is true, of course, that because the use of timber was less advantageous in much of Western Europe, than it was in the United States, bridges of other materials have always predominated.

The practice of assembling trusses on land before erection must have been more common than the authors imply. David Stevenson, a Scottish civil engineer, made an extensive tour of the Eastern United States and Canada, about 1837, for the sole purpose of inspecting engineering works.[55] He reported seeing many Town truss bridges on the Philadelphia and Reading, and on other, railroads.

He also noticed the Long bridge in many parts of the country, the best specimens being on some of the railways in the neighborhood of Boston, Mass. His account states, furthermore, that:

> "The timbers of which Town's and Long's bridges are composed, are fitted together on the ground previous to their erection on the piers. They are again taken asunder, and each beam is put up separately in the place which it is to occupy, by means of a scaffolding or centering of timber."

RICHARD S. KIRBY,[56] M. AM. SOC. C. E. (by letter).[56a]—The authors of this paper have covered in a thorough and interesting fashion a field too long neglected. The writer was especially pleased with their emphasis on personalities of the period. In this connection he ventures to add a word concerning the McCalls Ferry Bridge over the Susquehanna River, the work of Theodore Burr, Connecticut-born engineer and contractor, of Harrisburg, Pa. This bridge was one of Burr's great achievements, but little is heard of it because of the fact that it was carried down by the ice in 1817 after only two years' service. The writer believes that it had the longest span of any wooden bridge ever attempted in America, and that only that of the Schaffhausen Bridge had exceeded it.

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[55] "Sketch of the Civil Engineering of North America," Lond., 1838.

[56] Assoc. Prof., Eng. Drawing, Sheffield Scientific School, Yale Univ., New Haven, Conn.

[56a] Received by the Secretary January 13, 1933.

In a long letter written by him shortly after he finished this bridge, to his friend, Reuben Field, bridge builder of Waterford, N. Y., Burr rehearsed graphically his thrilling experiences in erecting the bridge during the winter of 1814-15. The arches were fabricated in halves and raised at first on bents of varying heights supported by floats ranged along and parallel to the shore nearly ¼ mile below the site. As the ice increased in thickness, Burr decided to use it instead of the floats. He finally slid the entire structure up stream on rollers. Man-power was abundant in those days, in that locality at least, for Burr commends the inhabitants for their willing help, which was perhaps gratuitous.[57]

WELLS N. THOMPSON,[58] ASSOC. M. AM. SOC. C. E. (by letter).[58a]—In the early stages of the development of wooden bridges, the crib or corbel type was one of the most interesting. Examples of this type of bridge are fast

FIG. 32.—OLD CORBEL TYPE OF BRIDGE, YAMANASHI PROVINCE, JAPAN.

disappearing as they are replaced by more modern structures, but there are a few remaining. Fig. 32 shows a very interesting one in the Far East.

This old bridge is well known throughout Japan, and it is shown or mentioned in many of their wood-block prints, poems, and songs. It is located in Central Japan, Yamanashi Province, and supplies its own name to the nearest town, "Saruhashi" or "Monkey Bridge." It is fifty miles west of Tokyo and spans a mountain stream at a point where it is approximately

[57] The letter appeared in *Niles Weekly Register* (Baltimore, Md.), November 18, 1815, It is copied in the Lancaster Historical Society Papers, Vol. XI, p. 133.

[58] Care, Thompson-Mitchell, New York, N. Y.

[58a] Received by the Secretary January 12, 1933.

80 ft wide between abutments. Age is evident in the entire structure, but repairs and replacements have been made as required so that it continues in good condition and in general use for the traffic in that district, which consists largely of pedestrians, jinrikishas, and ox-carts. The roadway is about 12 or 14 ft wide.

The Japanese excel in timber construction, and are very careful about protecting their construction against rot and decay. In this bridge, as in all other structures where framing is exposed to the weather, they have protected each member in the supporting cribs with an individual roof or watershed. Such care has preserved many of their very old and notable structures.

The abutments are field stone masonry, built against the rock walls, and have probably been added recently, in rebuilding or reinforcing the old stonework, perhaps after damage by earthquake.

The authors have presented an excellent treatment of this interesting subject.

BENJAMIN WILDER GUPPY,[59] M. AM. SOC. C. E. (by letter).[59a]—In connection with Fig. 28 of the paper, the authors point out the important warning that the bridge builder cannot be too careful as to designing the bolted and spliced joints of a timber bridge. A few examples should serve to emphasize this importance.

A draftsman who has had experience only in detailing metal structures has considerable difficulty when he first attempts to design a timber structure. In such a case the writer has told the man that if he designs a steel bridge while standing on his feet, he should stand on his head while designing a timber structure. In other words, the processes are reversed. In steel designing, the sections of the main members are determined first and then the details designed to transmit the stress properly. In straight timber work it is the design of the connection that generally determines the dimensions of the main members. In metal work the unit stresses to be considered are those of tension, compression, shear, and bearing. In timber the unit stresses to be accounted for are tension, compression, bearing on end grain, bearing across grain, shear parallel to fiber, and shear normal to fiber on planes parallel to and normal to direction of fiber.

The simplest case is a post in a trestle bent. In a steel structure the area of the post is determined by dividing the stress it is to carry by the allowable unit stress determined by some approved column formula. The dimensions of the timber post are determined by the stress in the post divided by the allowable stress in bearing across the grain on the cap or sill. A moment of reflection will show that it will be a very long slender post to require an allowable unit compressive stress as low as the unit bearing stress on side grain of timber.

The next step in advance is a king-post truss and to illustrate his statement the writer presents a design using timber entirely, use of iron being

[59] Engr. of Structures, B. & M. R. R., Boston, Mass.

[59a] Received by the Secretary January 23, 1933.

restricted merely to small bolts to hold the various pieces together (see Fig. 33). The king-post is in tension. To determine its proper dimensions both upper and lower connections must be considered. At the lower connection the size of the post is determined by the net area required in tension, plus the area required to transmit the stress by bearing across the grain. At the upper connection the area of the post is determined by the net area required to carry the tension, plus the area required to transmit the stress by bearing on the end grain. The larger of these areas governs. At the upper joint the strut must be framed both to give sufficient area for the vertical component in the strut at the unit stress for bearing on the end fiber and the horizontal component in the strut at the unit stress for bearing across the grain. The

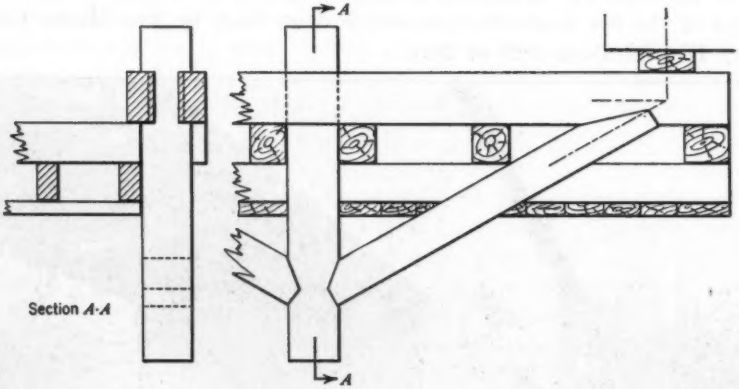

FIG. 33.

distance from the daps to the end of the king-post must be sufficient to prevent the timber from shearing beyond the daps. At the bottom of the strut the connection is reversed from that at the upper end as the horizontal component must be carried into the bottom chord through the end bearing and the vertical component through the side bearing. The bottom chord must extend far enough beyond the dap to give sufficient strength in the shear along the grain. The size of the bottom chord at the end joint is determined by the depth of dap necessary to transmit horizontal stress from the strut and the depth of stick required below the dap to carry the tension in the bottom chord plus any bending that may be induced by eccentric connections or methods of loading.

When it is necessary to splice a tension member the design of the splice is necessary before sizes of timbers can be determined. This is not submitted as an example of good modern practice, but to illustrate principles of design in timber and to show what will have to be done if metal for details is not available. The use of metal angle blocks to distribute stress at connections will permit of detailing compression members in accordance with an approved column formula and the use of rods for tension members and joint bolts will greatly simplify detailing connections. Figs. 16 and 28 in the paper by Messrs. Fletcher and Snow show how efficiency of tension splices can be increased by a free use of metal.

IVAN E. HOUK,[60] M. AM. SOC. C. E. (by letter).[60a]—The authors have contributed an extremely interesting and valuable account of the history, design, construction, and life of wooden bridges.

In connection with their discussion of retrieving weak and injured bridges, an account of the rather novel repairs shown in Fig. 34 may be worth while. This bridge is one of the old timber truss, covered highway bridges, still so common in the Eastern United States. It spans the Stillwater River, of Southwestern Ohio, about two miles above the Englewood Dam. It withstood, safely, the disastrous flood of March, 1913, as well as all the previous floods that had occurred after it was built. When it was built, or whether it is still in existence, is not known. However, it must have been about fifty years old at the time the Englewood Dam was completed. The Englewood Dam is one of the five large flood-prevention dams built by The Miami Conservancy District about 1920 or 1921.

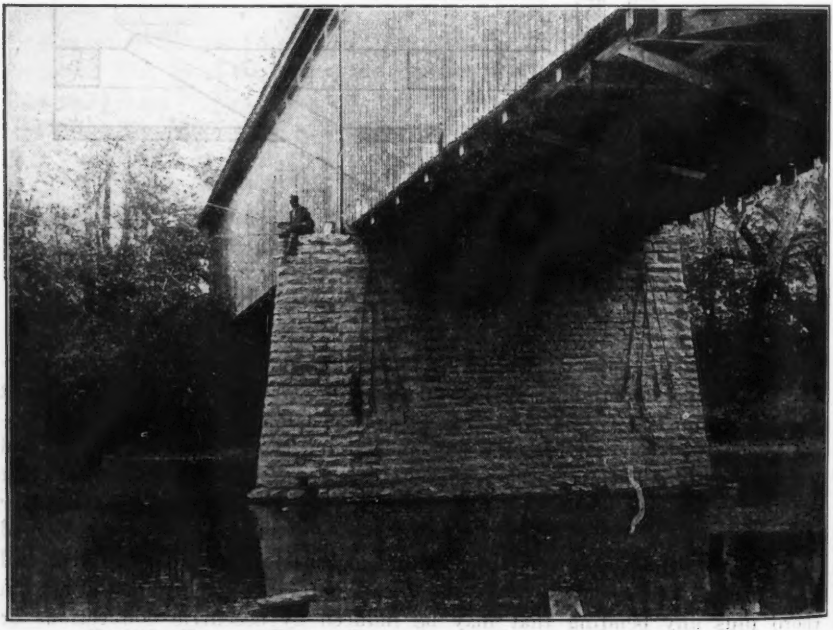

FIG. 34.—VIEW OF TIMBER TRUSS COVERED HIGHWAY BRIDGE OVER THE STILLWATER RIVER IN SOUTHEASTERN OHIO.

Prior to the completion of the flood-prevention works the bridge was located on the National Road and carried all through traffic along U. S. Highway 40 in that section of the State. Since that time through traffic has been routed over the top of the dam. However, the bridge was still in good condition at the time the dam was built, and it was desired to keep it in operation after the through traffic was diverted. Consequently, it was necessary to anchor it in place, to keep it from floating away during flood

[60] Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

[60a] Received by the Secretary January 23, 1933.

periods when the back-water above the dam might rise 30 or 40 ft above the roadway surface. In other words, the construction of the dam below made it necessary to hold the bridge down as well as up.

Iron rods, 1¼ in. in diameter, with an eye at the bottom and threads at the top, were placed vertically between the timbers of each truss, a rod being placed on each side of the pier and at each abutment. The upper end of the rod passes through the heavy timber which constitutes the upper chord and is held in place by a large iron washer and nut. The eye at the lower end is tied to the foundation by four steel cables, attached to iron bolts set in the masonry about 1 ft apart and about 12 ft below the lower chord.

The old, timber truss, covered bridges in the Miami Valley went through the destructive flood of March, 1913, as well as, if not better than, those of more modern design. Of course, this was because the masonry foundations had been either founded on rock or carried down to a sufficient depth below the bed of the stream, and had then been built to a height well above the maximum previous high-water level before placing the superstructure.

WILLIAM H. BAKER,[61] Assoc. M. Am. Soc. C. E. (by letter).[61a]—The various developments in the use of wood for the construction of bridges, is illustrated clearly in this paper. It covers the development from the most simple type, through all the stages of design, to the present standard. The writer believes that the development of the wooden bridge has been a great influence on the history of the world, both in military conquest and social growth through the medium of communication. The wooden bridge is still used extensively in the United States, despite the great progress in the use of steel and reinforced concrete in the last twenty years.

The wooden bridge is still used much more extensively than bridges of steel and reinforced concrete in Los Angeles County, California. There are approximately six hundred bridges of wood construction in use at the present time. They consist principally of A-frames up to 42-ft span, together with pony Howe trusses for longer spans. The reason for this extensive local use of the wooden bridge is due to the inadequate finances available to construct a more permanent type of crossing over the many dry washes in the county—most of them dry, except during flood stages—and also because as yet flood-control channels have not been established in many instances. Some of these wooden bridges over dry washes are as long as 800 to 1 000 ft, and to have built them of steel or concrete would have increased the cost many times that of the wooden structures; and after the flood-control channel had been established definitely, the channel would perhaps be much narrower than the original bridge span.

Since about 1925 or 1926 it has been necessary to widen many existing highways in Los Angeles County. The early bridges had 20-ft roadways—inadequate for present traffic needs. Most of the bridges now constructed have a roadway width which will be sufficient to provide for present traffic demands and for future widening of the highway. It was necessary, therefore, that the County Road Department adopt a policy in which all future

[61] Cons. Engr., Los Angeles, Calif.

[61a] Received by the Secretary January 25, 1933.

bridges would provide roadways of at least 30 ft. The wooden bridges, both the A-frame and the Howe truss type, were constructed using large Oregon pine floor-beam members. These floor-beans for a bridge without skew were 24 ft long for a 20-ft roadway, being made up of two 12 by 36-in. timbers bolted together. For skew bridges, the floor-beams were longer, as the angle of skew increased, making these beams from 24 ft to approximately 36 ft in length. It was very difficult to build wooden bridges for a 30-ft roadway utilizing wooden floor-beams, due to the length requirement of such a large wooden section. At the same time, the cross-sectional area required for the wooden floor-beams greatly reduced the head-room and opening for the waterway.

At the time that the Carnegie beam sections using flat flanges were introduced, the Bridge Department of Los Angeles County designed and developed a wooden truss bridge of the Howe type for 30-ft roadways, and for various angles of skew, utilizing the new Carnegie beam sections of 30, 36, and 42-in. depth for floor-beams replacing the old wooden type beams. A very simple method of constructing a proper connection for the hanger rods and extension brackets for the pony truss braces was designed and developed. This use of the Carnegie beam section as floor-beams, which has now (1933) been standard for five years, permitted the design and construction of wooden bridges of 30-ft roadway and a much greater angle of skew than was possible with wooden floor-beams. At a later date, about two years after steel floor-beams were utilized, the lower tension chord member was designed, with steel channels to replace the old, built-up, wooden type lower chord.

The principal difficulty experienced with bridges of the wooden type was the dry rot that developed in the end-posts and flooring. Several types of construction have been tried over a period of years to eliminate this, including the use of 2 by 6-in. timbers of Oregon creosoted pine, laid on edge and the utilization of diagonal planking overlaid with transverse planking at right angles and surfaced with asphalt or macadam.

It is impossible to state or give true results at the moment as to which type or method of wooden floor construction is superior. Later, due to the difficulty in maintaining the wooden type of floor construction, a reinforced concrete floor-slab system placed on timber stringers was tried as an experiment. Several bridges utilizing such construction have been in place for about four years, and to the present time, are in good condition. However, it is problematical whether this method will prove to be entirely successful and will replace the old method of wooden floor construction. The bridges that the writer last described are not strictly wooden, but are a combination of wood, steel, and concrete.

The use of the wooden bridge in Los Angeles County will probably continue for some years to come, until flood-control channels are definitely established and until new means of financing permanent structures of concrete and steel are advanced. The use of the wooden bridge is very costly, due to the heavy maintenance charges involved. However, they will have to serve until something more economical from the standpoint of first cost is substituted. This paper is of unusual interest and is a great contribution to civil engineering history.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

DISTRIBUTION OF SHEAR IN WELDED CONNECTIONS

Discussion

BY W. H. JAMESON, ASSOC. M. AM. SOC. C. E.

W. H. JAMESON,[12] ASSOC. M. AM. SOC. C. E. (by letter).[12a]—The formulas derived by the author when used in connection with the results obtained from actual tests reported in 1931 by the American Bureau of Welding[13] show rather surprising results. The author has proved mathematically that lengthening a weld beyond a very nominal length does not increase its strength. If tests proved this to be true, it would be disastrous for the welding industry and, for this reason, the writer feels that the fallacy in the author's assumptions should be shown. The writer has checked the derivation of all the formulas and, if the fundamental assumptions are accepted, has found the actual mathematical work correct. Therefore, the error must be in the assumptions since a true mathematical solution must check the results obtained from tests.

Mr. Troelsch states that the stress in Bar 1, Fig. 1, is zero at A and increases to a maximum at B; and that, similarly, the stress in Bar 2 is zero at B and maximum at A. At some intermediate point, O, therefore, the unit stresses in the two bars will be equal, and the bars will have equal deformation. This assumption implies that the stress at any right section across either Bar 1 or Bar 2 is uniform throughout the bar. To the writer this seems unfounded. The bars are loaded eccentrically with forces along the weld at the edges and the probable deformation of the bars might be represented as shown in Fig. 10.[14]

Fig. 1 (*e*) is the elevation of a section of the two bars, which shows actual slipping between the two bars at the weld. Actually, the bars cannot slip at the weld without tearing it so the value on which practically the entire solution is based, is incorrect. At the section, O, the stress in the two bars

NOTE.—The paper by Henry W. Troelsch, M. Am. Soc. C. E., was published in November, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1933, by Messrs. F. T. Llewellyn, A. S. Woodle, Jr., Milton Male, William Hovgaard, Charles W. Chassaing, and F. E. Fahy.

[12] Designer, McClintic-Marshall Corp., Bethlehem, Pa.

[12a] Received by the Secretary January 10, 1933.

[13] Rept., Structural Steel Welding Committee, September, 1931.

[14] *Loc. cit.*, p. 100.

is the same only at the actual point of contact of the two bars and the weld. At any other point on a right section at O, the stress is different from that at the point of contact defined herein.

A study of the results of tests given in the report referred to, shows that for a given size of weld, the ultimate stress on the throat of the weld, based on

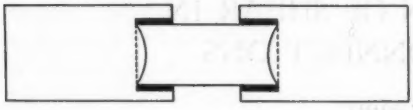

FIG. 10

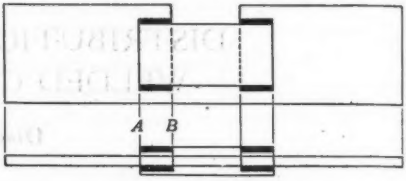

FIG. 11

the total effective length[15] averaged about 38 000 lb per sq in. It is somewhat higher (up to 39 500 lb per sq in.) for short welds and somewhat lower for long welds (37 500 lb per sq in.). This would tend to prove that probably the author's solution has some basis in fact, but since the difference is so small between long welds and short welds, it may be neglected for all practical purposes. For the same specimens, the maximum shear values by Equations (17) and (18) show wide divergence. The results of a few tests are given in Table 2 and all others tested by the writer using the author's formulas show

TABLE 2.—ANALYSIS OF TESTS ON WELDED SPECIMENS.

Test No.	Length of weld, z, in inches	Average ultimate strength of weld, p, in pounds	Shears, in Pounds per Square Inch		
			Average on the weld, $\frac{P}{0.265z}$	Maximum, by Equations (17) and (18), Divided by 0.265 $\left(\frac{E}{D}=2\right)$	
				At Section A, Fig. 11	At Section B, Fig. 11
(1)	(2)	(3)	(4)	(5)	(6)
2333.....	7	69 200	37 300	50 300	54 000
2433.....	23	241 000	39 500	95 500	97 500
2443.....	31	319 000	38 800	99 000	114 400
2463.....	47	468 000	37 500	129 300	132 000

similar results. All these tests were made on specimens with $\frac{3}{8}$-in. fillet welds made up as shown in Fig. 11. In Column (3), Table 2, the total ultimate strengths have been reduced to a common basis of a throat thickness of 0.265 in. before averaging. Columns (4), (5), and (6) show similar values for the average shear, and widely divergent values for the maximum shears by the author's formulas. Since it is unlikely that failure would occur at such different values as the author's formulas would indicate, the writer feels that for welds of ordinary length (certainly up to 12 in. and probably up to at least 24 in.), it is perfectly safe to use the average shearing stress over the entire effective length of the weld. This is the method commonly used and there seems to be no reason to change it, or to fear that welded joints designed by this method will prove to be unsafe.

[15] Rept., Structural Steel Welding Committee, September, 1931, p. 45.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

WORK OF RIVETS IN RIVETED JOINTS

Discussion

BY MESSRS. HENRY W. TROELSCH, HENRY B. SEAMAN, A. H. FINLAY, AND F. P. SHEARWOOD

HENRY W. TROELSCH,[3] M. AM. SOC. C. E. (by letter).[3a]—The problem of the distribution of stress in riveted joints, which Mr. Hrennikoff considers in this paper, is similar in many ways to the problem of the distribution of shear in welded connections, dealt with by the writer in the companion paper.[4] The two are in fact special cases of a more general problem. The only difference is that, for the riveted joints, the connection between the spliced members is assumed to exist only at uniformly spaced points (the rivet centers), while for the welded joints, the connection is assumed to extend uniformly from one end of the joint to the other. In both cases it is assumed that the deformations of the bars and of the connecting members (rivets or welds) are proportional to their stresses.

The similarity of the two theories can be shown by applying the weld theory to several of the riveted joints analyzed by Mr. Hrennikoff, and comparing the results. Let each rivet be replaced by a single weld, equal in length to the pitch, and having the same rigidity as the rivet. The length, z, of the equivalent welded joint will then be the number of rivets multiplied by the pitch. The "detrusion ratio", D, becomes $\frac{1}{kp}$. The shears per unit of length at the rivet centers can then be obtained by Equation (15). These shears multiplied by the pitch will give approximate values of the forces acting on the rivets.

The results for four cases are shown in Table 2, in which the values marked "Weld Theory" were obtained as indicated herein, while those marked "Rivet Theory" are the corresponding coefficients taken from Mr. Hrennikoff's paper. The close agreement between the two theories is brought out

NOTE.—The paper by A. Hrennikoff, Esq., was published in November, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[3] Designing Engr., Am. Bridge Co., New York, N. Y.

[3a] Received by the Secretary December 7, 1932.

[4] "Distribution of Shear in Welded Connections," *Proceedings*, Am. Soc. C. E., November, 1932, p. 1499.

TABLE 2—COMPARISON OF DISTRIBUTION COEFFICIENTS, C, AS DETERMINED BY WELD THEORY AND RIVET THEORY

Number of rivets	Width of strip, w, in inches	Pitch of rivets, p, in inches	Thickness of plate, t, in inches	Rivet coefficient, k, in inches per kip	Detrusion ratio, D, in kips per square inch	Length of equivalent weld, z, in inches	Distribution Coefficients			
							Theory	C_1	C_2	C_3
6.....	3	2.5	1	$\frac{1}{6\,400}$	2 560	15	Weld...	0.239	0.146	0.107
							Rivet..	0.241	0.149	0.110
6.....	3	5	0.5	$\frac{1}{8\,200}$	1 640	30	Weld...	0.344	0.0907	0.0291
							Rivet..	0.3596	0.1036	0.0368
3.....	3	2.5	1	$\frac{1}{6\,400}$	2 560	7.5	Weld...	0.347	0.293	.....
							Rivet..	0.351	0.298	.....
3.....	3	5	0.5	$\frac{1}{8\,200}$	1 640	15	Weld...	0.373	0.181	.....
							Rivet..	0.395	0.210	.....

even more clearly in Fig. 6, which shows the same four cases. The ordinates to the dotted curves are the shears per inch in the equivalent welds, while the ordinates to the plotted points are Mr. Hrennikoff's rivet coefficients divided by the pitch. Fig. 6 shows that if the assumptions underlying the

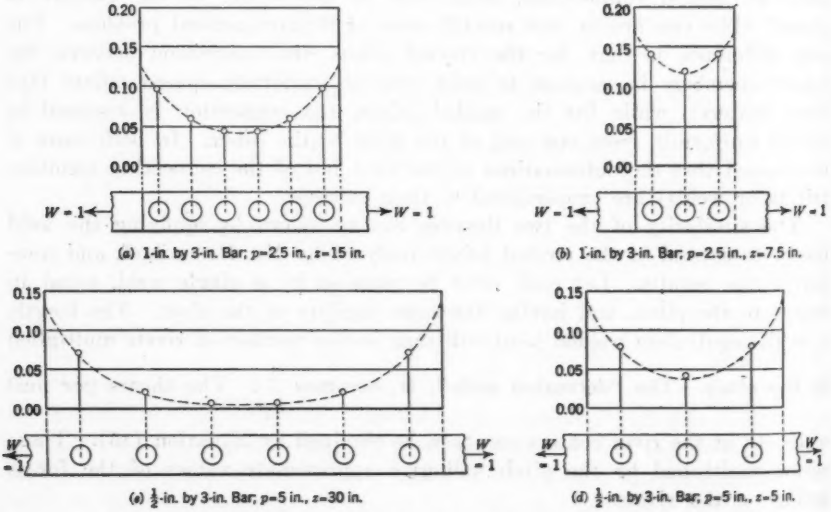
0.20
0.15
0.10
0.05
0.00
W = 1
W = 1
(a) 1-in. by 3-in. Bar; p=2.5 in., z=15 in.
(b) 1-in. by 3-in. Bar; p=2.5 in., z=7.5 in.
(c) ½-in. by 3-in. Bar; p=5 in., z=30 in.
(d) ½-in. by 3-in. Bar; p=5 in., z=5 in.

FIG. 6.—AGREEMENT BETWEEN WELD THEORY AND RIVET THEORY.

theories are fulfilled, the distribution of the shear follows the same general law, whether the joint is riveted or welded.

HENRY B. SEAMAN,[5] M. AM. SOC. C. E. (by letter.[5a])—The analysis presented by Mr. Hrennikoff treats of a subject that has long claimed the attention of designers of riveted trusses, and explains the preference of many engi-

[5] Cons. Engr., Brooklyn, N. Y.

[5a] Received by the Secretary November 21, 1932.

neers for the pin-connected type, even for the shorter spans. In the earlier days of American bridge construction the pin-connected type was used for spans as short as 80 ft., and gave great satisfaction in maintenance; but the preference of the shops for less labor, with heavier tonnage and correspondingly lower shop costs, brought the riveted truss into its present favor.

It was in 1886 that the matter first claimed the attention of the writer, in a riveted lattice girder of about 50-ft. span, for an elevated railway. There were five rivets in the joint, and the writer preferred to place the odd rivet in the center of the cluster, leaving two end rivets to take the initial strain. When the matter was referred to the late Theodore Cooper, M. Am. Soc. C. E., however, he preferred that the single rivet be placed on the outside so as to save material by the cutting of one hole, instead of two holes. This well illustrates the difference between first cost and subsequent cost of maintenance.

The objectionable differences in the stresses of the rivets of a joint, explained by Mr. Hrennikoff, do actually exist in all riveted joints.

A. H. FINLAY,[6] ASSOC. M. AM. SOC. C. E. (by letter)[6a]—In this paper, Mr. Hrennikoff has done a service in calling attention to the weaknesses of the assumptions underlying the common analysis of riveted joints. A periodical review of the assumptions that underlie various structural theories is of the greatest importance, since it is a human failing that, given sufficient time for their assimilation, assumptions at first viewed even with some misgiving eventually tend to be confused with the eternal verities.

In Part I the author has given a clear discussion of the probable behavior of rivets in a joint. Part II translates into the more imposing symbolism of mathematics the assumptions of Part I for the specific case of no friction between the plates.

The wisdom of omitting from consideration the friction between the plates will scarcely be challenged. To any one who has watched rivets being driven with a so-called "bull-riveter" and who has seen the air pressure removed, as is commonly done, within 2 or 3 sec of its first application, the stretching of the still incandescent rivet shank by the spring of the plates upon removal of the pressure, is full of significance.

The writer feels that special credit is due the author for his mathematical evaluation of the proportionality factor expressing the ratio of rivet slip to rivet stress, or k, without which his results would have been merely interesting qualitative relationships. The lack of specific values of this quantity has greatly handicapped previous work in this field. As to the correctness of the author's two values for k, one can only expect that they approach the truth as closely as do the very necessary assumptions, under cover of which the obviously difficult mathematical problem was attacked. The author wisely omits the analysis and modestly refers to it as "rather long."

The writer agrees that the determination of k is a proper field for experiment, and he would be interested to hear of any tests in this connection,

[6] Asst. Prof., Civ. Eng., Univ. of British Columbia, Vancouver, B. C., Canada.
[6a] Received by the Secretary January 3, 1933.

although he feels that, in practice, the values of k are as divergent as the conditions of driving.

The general satisfaction given by riveted joints as at present designed as well as the simplicity and economy resulting from them are sufficient guaranties that no radical changes in design will be made. It would be unwise to think of penalizing structural steel for one of its greatest virtues —ductility. How many discordant elements this quality reconciles may never be known, but structural engineers have good reason to appreciate its existence.

With regard to the author's several suggestions for the improvement of riveted joint design, the writer feels that, while in all joints a more uniform distribution of stress will result among the rivets, there is danger of a poorer distribution of stress occurring across the gusset-plate, owing to the more compact rivet groups and the general tendency to bunch rivets which is implied.

In closing, the writer feels that the profession is fortunate in having two such papers—this one concerning rivets and one, by Henry W. Troelsch, M. Am. Soc. C. E., concerning welded joints—published simultaneously.

F. P. SHEARWOOD,[7] M. AM. SOC. C. E. (by letter).[7a]—This paper deals with one of the many problems encountered in structural design whenever riveted joints are closely investigated. They often worry the designers, but by general consent the elasticity and ductility of the joint are depended upon to distribute the strain safely. While such easy method has been recognized and used, it would be more satisfactory to understand and picture what the strains are likely to be, so that designers can proportion and group the rivets and material to obtain the greatest safety.

Mr. Hrennikoff has given a clear and useful theory for calculating the rivet strains. His theory is based on perfect elasticity throughout the entire joint. If the initial resistance is from friction, however, and if there is some clearance in the rivet hole, the distribution will not be truly elastic and, when relieved of its load, the joint will not completely recover.

Although the author has referred to friction between the plates, he has disregarded it in his analysis. This may be of the greatest importance, however, in preventing over-stressing rivets in long joints. If there is any clearance between the rivet and its hole, and frictional resistance is about equal to the working value for single shear, the stress at any rivet can not exceed the working value, until the clearance is absorbed by slipping.

The author states that long joints are constantly being used and that none of them has failed. This fact has been proved so often that it has become standard practice to assume that the stress in a joint is equally distributed to all the rivets; that is, that the strength of a riveted joint is the sum of the individual rivet values. It is the general opinion, however, borne out by experience, that connections resisting severe reversals of stress will become loose, and, therefore, must be proportioned differently. These facts indicate that there is some straining effect that is not elastic and that

[7] Chf. Engr., Dominion Bridge Co., Ltd., Montreal, Que., Canada.

[7a] Received by the Secretary December 16, 1932.

produces wear; and it clearly points to a slip taking place between the plates, or between the plates and the rivet heads. Frictional distortion or slip of the plates caused by stress in the member will not recover on the removal of the stress, unless there is as great a difference between the strains in the plates as that which caused the slip.

To allow slip to take place without shear in the rivets there must be clearance between the rivet and the hole. That this exists seems very probable. When a rivet is driven its heat temporarily reduces the size of the hole. The rivet is upset to the size of the heated hole, and pressure is released from both the hole and the rivet when they are at a high heat. Both the hole and the rivet must cool, and, therefore, a clearance is likely. As an illustration, let it be assumed that when the driving of a rivet is completed there is an average temperature of 500° above normal temperature in the edges of the hole at the shank of the rivet; then the clearance for a $\frac{7}{8}$-in. rivet will be approximately: $0.0000065 \times 500 \times 2 \times 0.875 = 0.00525$ in. That means that the plates can move past each other 0.005 in. before the rivet touches the walls of the hole and it is strained in shear or bearing.

The amount of frictional resistance will depend on the initial tension developed in the rivet by cooling. It seems probable that this tension on the rivet head is about equal to the elastic limit of the material, say, 35 000 lb per sq in. By assuming the coefficient of friction at 20%, the resistance at each surface will be $35\,000 \times 0.6 \times 0.2 = 4\,200$ lb.

To enable the plates to move, at least two surfaces are affected; therefore, the force required to overcome friction will be equal to about 8 400 lb per rivet. Fig. 7 illustrates the clearances, relative positions, and strains in the holes of an 8-rivet lap-joint, under various conditions of loading. The joint is composed of two 1-in. by 3-in. plates. In order to simplify the computations, the force required to overcome friction is assumed equal to 7 500 lb per rivet, or 2 500 lb per in. of width, and $E = 30\,000\,000$ lb per sq in. The plates are shown extended an amount equal to 0.00033 in. per 10 000-lb strain, or 0.001 in. per 10 000-lb strain between rivets. Stresses are given in pounds per square inch of material. Fig. 7(*a*) illustrates the clearances in the holes to a very exaggerated scale. The total clearance between the hole and rivet is assumed to be 0.0025 on each side, or 0.005 in all.

Fig. 7(*b*) gives the relative positions and strains of the two plates when a stress of 20 000 lb per sq in. of the main plate is applied (full section is assumed at holes). When this load is applied, the frictional resistance of one rivet will be exerted at Rivet *a*, but as this is insufficient to equalize the strain in the two plates they will slip past each other, and a load of 17 500 lb per sq in. will remain in the upper plate until the resistance of Rivet *b* is met, where another 2 500 lb per lin in. (the resistance from one rivet) will be transferred to the lower plate, and so on, until the strains in the two plates are equal.

The difference of deformation in the upper and lower plates between the point where the strains are equal and any rivet in the connection will give the amount of slip at that point. In this example, the slip at Rivet *a* equals 0.003 in., leaving a clearance of 0.001 in. between the hole and rivet. At

Center of Weld
Rivet
(a) NO LOAD (BEFORE STRESSING)
(b) LOAD OF 60 000 LB (20 000 LB PER SQ IN.)
(c) LOAD OF 30 000 LB (10 000 LB PER SQ IN., AFTER STRESSING TO 20 000 LB)
(d) NO LOAD (NO EXTERNAL LOAD, AFTER STRESSING TO 20 000 LB)
(e) LOAD OF 75 000 LB (25 000 LB PER SQ IN.)
Clearances in Holes Shown Hatched
Scale of Clearances
0.01 in.
No Clearance
Nil
-20 000 lb per sq in.
-10 000 lb per sq in.
-25 000 lb per sq in.

FIG. 7.—CLEARANCES, RELATIVE POSITIONS, AND STRAINS IN HOLES OF AN 8-RIVET LAP-JOINT.

Rivet *b*, it is 0.0015 in., and at Rivet *c*, 0.0005 in., while at Rivet *d*, there is no slip because the strains in the plates are equal. Some clearance still remains at all these rivets and, therefore, no shearing can have been developed in the rivets themselves. The bending resistance in the rivet due to its heads slipping on the plates cannot be very much. If it is disregarded there can be no stress of any kind in the rivet, until the slip of the end rivet has exceeded the clearance in the hole.

Fig. 7(*c*) illustrates the positions and strains when 50% of the stress is removed. No alteration in the relative positions of the two plates is necessary to keep the friction below the slipping value of 2 500 lb and, therefore, no slip or wear results. When more than 50% of the load is removed the plates must slip to reduce the friction to 2 500 lb. Fig. 7(*d*) shows the positions and strains resulting when all the stress is removed, and it shows also that Rivets *a* and *b* have partly slipped back. The 20 000-lb unit strain assumed in this example does not often occur in actual practice, and it is probable that the ordinary range of strains developed will not be sufficient to slip the plates back and loosen the rivets by wear.

When the stress exceeds the frictional value of all the rivets the entire connection will slip until the end rivets come into contact with the walls of the holes. These end rivets must resist the excess of stress over the friction value of all the rivets until the deformation of these end rivets is sufficient to bring the next rivet into contact.

Fig. 7(*e*) gives the clearances in the holes when this connection is stressed to slightly beyond the total friction value of the eight rivets. The rivets at each end must transfer 5 000 lb per sq in., which is twice the assumed friction value of the rivet friction; but there is still clearance at the second rivets. (No allowance has been made for the deflection from shearing and bearing of the end rivets.)

For the sake of simplifying the general picture of slippage, these examples of lap-joints are based on the assumption that the rivet heads slip on the plates at one-half the resistance and displacement that the plates slip past each other. The resistance from the bending of the rivet is also neglected.

If a butt-joint is examined it will be found that the same general slipping takes place, except that the plates will first slip on one another until the rivet shank bears against the inner plate. The heads will then slip in the outer plates, and two extra slipping surfaces will be brought into resistance, until the end rivet is jammed between the three plates and any further resistance must be entirely taken up by the end rivets in shear until the deformation is sufficient to bring the next rivet to bear on the walls of its hole.

Fig. 8 is a corresponding example of a butt-joint. The force required to overcome friction is assumed to be equal to 3 750 lb per rivet on each surface, or 1 250 lb per rivet on each surface per inch of width. As in Fig. 7, stresses are given in pounds per square inch of material. It can be readily seen that the friction between the plates at all the rivets and the friction between the rivet heads and plates at Rivets *a* and *d* are just sufficient to resist the load, while at Rivets *b* and *c* some clearance still remains. Any addition to the load will be entirely taken by Rivets *a* and *d* in shear until the deformation

FIG. 7.—CLEARANCES, RELATIVE POSITIONS, AND STRAINS IN HOLES OF AN 8-RIVET LAP-JOINT.

of those rivets amounts to the clearances remaining at Rivets *b* and *c*. In the case of the butt-joint the resistance to recovery when the load is removed is greater than in the case of the lap-joint.

The stresses and dimensions of clearances are only approximate and are used to illustrate, in the simplest way, the possible workings in a riveted

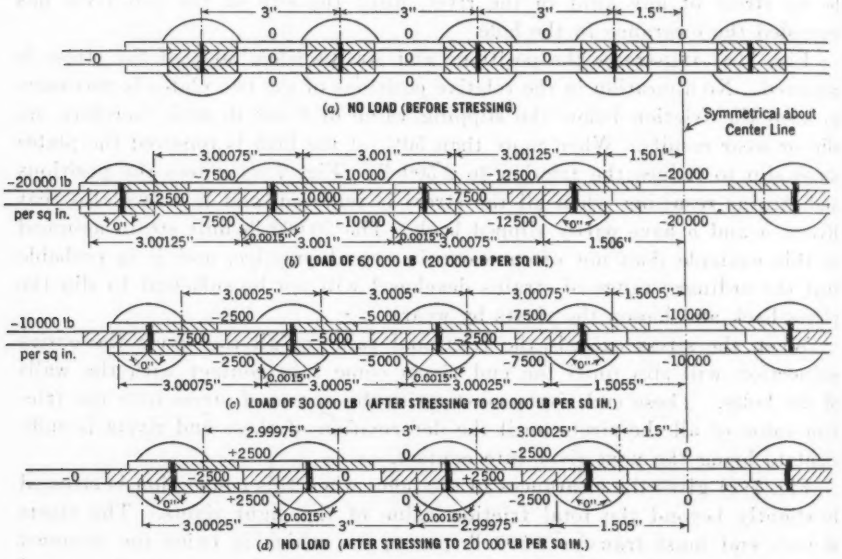
(a) NO LOAD (BEFORE STRESSING)
Symmetrical about Center Line
(b) LOAD OF 60 000 LB (20 000 LB PER SQ IN.)
(c) LOAD OF 30 000 LB (AFTER STRESSING TO 20 000 LB PER SQ IN.)
(d) NO LOAD (AFTER STRESSING TO 20 000 LB PER SQ IN.)

FIG. 8.—CLEARANCES, RELATIVE POSITIONS, AND STRAINS IN HOLES OF AN 8-RIVET BUTT-JOINT.

joint, which has minute clearances in the holes when under stress. They also illustrate the fact that very unequal distribution of stress among the rivets of a group is not probable in the usual length of riveted connections.

The problem of how much the frictional resistance amounts to and what range of stress can be used without producing constant slip, depends on a great number of variable uncertainties and would be difficult to solve by mathematics. Experience indicates that the range can be almost up to reversal without producing wear, and general practice has largely assumed this in designing riveted structures. The main problem is, as to whether there is any clearance in the rivet holes, and, if so, how much?

The writer has endeavored to show: First, that there is clearance between the hole and the rivet and that this clearance must be absorbed before the rivets can take any shear; and, second, that slipping of the connected plates occurs and that the force to produce this movement is consistent; that is, it does not vary with the amount of movement. If these are the facts and the deformation required to equal the clearance is sufficient to absorb the total strain in joints of usual length, Mr. Hrennikoff's theory, which is based on the elasticity of the rivets and material, does not function, except for a small part of the stresses in very long joints.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

MODEL LAW FOR MOTION OF SALT WATER THROUGH FRESH

Discussion

BY W. E. HOWLAND, ASSOC. M. AM. SOC. C. E.

W. E. HOWLAND,[4] ASSOC. M. AM. SOC. C. E. (by letter).[4a]—This study seems to be a novel and most promising application of the methods of model testing to a baffling and important problem. It would seem to suggest methods for the study of other related problems, such as the spread of sewage over the surface of a salt-water harbor already treated[5] by A. M. Rawn and H. K. Palmer, Members, Am. Soc. C. E., and others.

Referring to the authors' Fig. 6, and considering as a free body that portion of the fluid between a section taken a little to the left of Section *C-C*, and a section taken a little to the right of Section *E-E*, the horizontal forces acting on this free body would be $\frac{1}{2} s w d^2$, toward the right (if the levels are as shown) and also friction forces the resultant of which probably acts toward the left. Are these forces necessarily in equilibrium? Is it not possible that they have a resultant acting to the right that tends to move the entire body in that direction, tending thus to depress the water surface at *C* and raise it at *E*? If this is not so, it is difficult to understand how the fresh water between *C* and *E* can start to move toward the left notwithstanding the authors' statement that the only surface disturbances observed were those directly attributable to the opening of the gate.

Does the interface of the salt and fresh water incline as shown in Fig. 6? If so, one can understand how the pressure gradient in the salt water might be toward the right, imparting motion in that direction. If, however, this interface slopes toward the left, as suggested in Fig. 7, it would seem necessary to assume a considerable decrease of momentum in the direction of motion. Possibly an assumption of mixing that would have the result of

NOTE.—This paper by Morrough P. O'Brien and John Cherno, Assoc. Members, Am. Soc. C. E., was published in December, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[4] Asst. Prof., Civ. Eng., Purdue Univ., Lafayette, Ind.

[4a] Received by the Secretary January 6, 1933.

[5] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1036.

clouding the effects, could account for the appearance indicated. If the former assumption is correct, it would seem that the condition illustrated in Fig. 7 would be a secondary one and would not be the condition prevailing at the beginning of the motion. In order to make the salt water continue to move to the right, it would seem that there must be a pressure gradient predominating toward the right within the salt water. Fig. 7 illustrates a most interesting condition, in which all the exterior forces on the moving salt water are tending to slow it down. This might explain the rapid decrease in velocity observed in certain of the experiments.

If the writer's assumptions as to the depression of the water surface near C and the corresponding rise near E, are correct, it should be possible to detect a movement of the surface water toward C in both directions and away from E in both directions. Were any such movements detected? Furthermore, it might be supposed that such changes in elevation of surface would give rise to waves of small amplitude which, in time, would cause a jerky or intermittent motion of colored salt water. Was this effect noticed? Messrs. Rawn and Palmer[6] recorded as an observation of their experiments that "colored fresh water moved over the salt water in concentric waves"; but, of course, the conditions in these two experiments are entirely different.

Similar considerations would suggest that when the gate is opened only a slight amount at the bottom, as indicated in Fig. 8, it is possible that the water would move entirely in one direction—that is, toward the fresh water—and that no fresh water would move in the other direction. Therefore, the initial velocity toward the fresh water might be even greater than as computed; that is, it might be nearly as great as $\sqrt{2g}\sqrt{\frac{ds}{G_1}}$. Have any direct observation of this case been made?

The writer has obtained the model law (Equation (16)) by methods which he believes to be similar to those used by the authors, by assuming that: (*a*) The impressed forces on a body between Sections C-C and E-E (Fig. 6) are proportional to $w\,s\,d^2$; (*B*) the resistant forces are proportional to vd, x, *or* L, and kinematic viscosity; and (*c*) the inertia forces are proportional to $\frac{w}{g}$, L, d, and $\frac{v^2}{L}$. With these assumptions he has written the equation which states that the ratio of these three pairs of corresponding forces must be the same in the model as in the prototype. The assumption of constant viscosity is also made.

When another pair of resisting forces is introduced, assuming that these forces vary indirectly with the velocity squared, v^2, the area of the stream, $d\,L$, and, inversely, as the hydraulic radius assumed to be proportional to d, another limitation is obtained which states that $L = d$; and then the model law becomes,

$$\frac{1}{d^{1.5}\,s^{0.5}} = \text{a constant} \qquad (18)$$

[6] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1047.

Thus, for the similarity of all these forces, a scale ratio of $\sqrt[3]{10}$ would be required when the salinity ratio is 10, which is too large to be of much practical value. Fortunately, the authors' experiments have clearly shown that for their problem this additional condition need not be met.

There is one other effect which, under certain conditions, might modify the model law and should receive further study before the law is given a wide application. This effect is mixing. Consider a particle in the fresh water moving over the top of the body of salt water. The impressed force in this case is proportional, not only to the surface gradient (which, in turn, is dependent upon the depth, d), but also upon the density gradient, because both affect the resultant pressure in the direction of motion. The density gradient, of course, is dependent upon mixing. If it can be shown that adherence to the model law will automatically give density gradients either negligible or proportional to the surface gradient, or if, in some other way, it can be shown that all the corresponding important forces are in proper ratio, then the objection just raised would have been overcome. The consistency of the experimental data presented does, in a measure, prove this for the case studied, but any information on the degree of mixing or of salinity that may have been obtained in these experiments, would be a most useful addition to the paper.

As an illustration of the possible effect of mixing upon the model law, let it be assumed that the resistance offered to the motion is dependent, not only upon the area of contact between the two moving fluids, L, the velocity gradient, $\frac{v}{L}$, and the viscosity, but also to some extent upon the differences in density between the two fluids, s. This is reasonable since it is known that the mixing is the greater, the less the difference of the density of the fluids; and, of course, the greater the mixing of the two oppositely moving fluids, the greater the resistance to their motion. The late Kenneth Allen, M. Am. Soc. C. E., has expressed[7] this idea in the statement: "It is clear that salinity of a body of water receiving sewage is an important factor in its diffusion." Let it be assumed that $\frac{1}{s^n}$ should be introduced into Equation (18). Then, it can be shown that the model law would be changed to,

$$\frac{d^{2.5}\ s^{(0.5\,+\,0.5n)}}{L} = \text{a constant} \ldots\ldots\ldots\ldots\ldots\ldots (19)$$

in which, n is an unknown, but positive, exponent.

In the case studied by Messrs. Rawn and Palmer, the mixing effect was a significant factor. Indeed, its evaluation was the object of their study, and they concluded that no stratification at all is possible if the dilution exceeds 1:200.

Does it not seem plausible that the more rapid decrease in velocity observed for large values of k (when both d and s are small) may have been occasioned by the greater mixing that takes place under those conditions? The

[7] *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 1065.

effect of a small, s, upon mixing has just been considered. A small d would give rise to large velocity gradients which, in turn, would seem to promote mixing. Since the forces arising from turbulence have been negelected, in deriving the model law, it would seem that the necessity of providing turbulent conditions in both model and prototype would arise from the requirement of producing similar mixing effects. Surely, if the effect of mixing is important, then immiscible liquids, as mentioned under "Conclusions," could not be used in the model.

The validity of K as a criterion for the case studied would be more convincing if the authors would show the actual computed values for several points in Fig. 7, so that it would be possible to prove from actual data that for radically different values of s and d, with identical values of K, the points as plotted would be substantially coincident.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

SOCIETY AFFAIRS

ANNUAL REPORT OF THE BOARD OF DIRECTION FOR THE YEAR ENDING DECEMBER 31, 1932

In compliance with the Constitution, the Board of Direction presents its Report for the year ending December 31, 1932.

THE EIGHTIETH YEAR

Eighty years ago, on November 5, 1852, a small group of Civil Engineers met in the office of Alfred W. Craven, then the eminent Chief Engineer of the Croton Aqueduct for the City of New York, and organized the "American Society of Civil Engineers and Architects." The objects of the new Society were declared to be: "The professional improvement of its members, the encouragement of social intercourse among men of practical science, the advancement of engineering in its several branches, and of architecture, and the establishment of a central point of reference and union for its members." In the eighty years that have followed those ideals have changed but slightly in phraseology and little in intent. The corporate situation, however, has changed markedly. Then, the Society was a small and hesitant group, largely of New York City residents. Now, it is a sturdy aggregation of more than 15 000 members situated throughout the world. This report, covering the eightieth year of the Society's experiences, records a present membership of 15 247; an expenditure of $347 112; and proportionate intensification and diversification of activities.

Growth of membership, with consequent increase of income, was particularly marked in the years 1925 to 1931. In proportion, new facilities for service to members were practicable and new departments were developed, organized, and expanded. It may be said that the year 1930 was the peak year of the Society's history. The year now closed, the eightieth year, as a result of the long continued restricted business conditions and in common with practically every membership organization, has brought to these activities the necessity for re-adjustment and curtailment. Thanks to the loyalty of the membership and the fortunate financial condition of the Society, the re-adjustments have been necessary in only relatively minor degree. No units or fields of endeavor have been abolished, although practically all have been curtailed somewhat.

Finances

The principal source of income consists of the dues from members, and it is in this item that the greatest reduction in revenue has occurred. Income from this source diminished $36 649 from last year's figures and $47 669 from those of 1930, respectively, losses of 15.1 and 18.7 per cent. Other sources of income, for services rendered, fell off from both 1930 and 1931, but these in the main were counterbalanced by a proportionate decrease in related expenditures. The rent received from the 57th Street Property, a considerable item, fortunately has remained unchanged. The necessary curtailments in activities, therefore, all seem to be attributable largely to the decrease in dues collected. Publications were reduced in volume; the Spring Meeting was omitted; mileage allowance was reduced; salaries of the Staff were reduced; committees were asked to carry on so far as possible, after June 1, by correspondence and without the expense of meeting; contributions to the Local Sections, to American Engineering Council, and to Engineering Societies Library were decreased. Expenditures, however, were not curtailed in proportion to the falling off in the amount of dues collected. The budget was adopted, and the ratio of expenditures to income throughout the year was maintained consistently on the basis that an amount deemed to be reasonable was to be available from previous accumulation of excess of income over expenditures. The year closed with a withdrawal from reserves of $8 572. Expenditures were 16.7% less than those of 1931.

Membership

Membership at the close of the year was 57 greater than at the same date last year, thus maintaining the record that never has there been a year in which the Society's membership has not increased. The situation among the members, however, is much more serious than these figures would indicate. Unemployment among engineers has been very great and very long sustained and the members of the Society have been by no means exempt from this experience. Were the prompt payment of dues to be the sole criterion, the figures denoting members in good standing would be approximately 2 000 less than recorded herein. Prompt payment of dues, however, is not the only criterion. The Society throughout its eighty years has built up a selected group of members, loyal to the Society and its traditions, and ready to contribute of their time, their efforts, and their capabilities to the advancement of the profession. The value of the Society's members as men is far greater than the value of their dues as money. This the Board of Direction has recognized and 2 033 members who have supported the Society in years past, some twenty years, some thirty years and more, but who in these times are unable to pay dues, are continued, as "in good standing" to-day without such payment—continued as an asset in men, not money.

Relief Work

Definite recognition should be made of the work done by the Local Sections of the Society in the extension of financial relief to those engineers in need thereof, whether or not members of the Society. This general procedure was recommended to the Board of Direction by the Society's Committee on Salaries in October, 1931. Upon adoption by the Board a central bureau was established in the Headquarters Staff. The outstanding instance of relief work is afforded by the New York Metropolitan Section. At the suggestion, and with the very energetic leadership of the President of that Section, there was formulated a program jointly with the Metropolitan Sections of other Founder Societies. This program resulted in providing unemployed engineers, both members and non-members, with work for which they received $874 386.37 in compensation in the period from November 1, 1931, to October 1, 1932. Work, similar in purpose, has been carried on by many of the Local Sections in other communities.

Public Works

Undoubtedly, the most outstanding instance of new activities undertaken during the past year is the "Normal Program for Public Works Construction to Stimulate Trade Recovery and Revive Employment," approved by the Board in principle and by the Executive Committee by resolution. A Committee on Public Works, after continued practical efforts addressed to the Congress of the United States, saw the proposed program incorporated in the Federal legislation entitled "The Emergency Relief and Construction Act of 1932."

Under this Act the Reconstruction Finance Corporation is empowered to make loans for certain construction projects, and the members of the Committee continued their efforts through that Corporation to make the purpose of the Act as widely effective as possible. Toward the close of the year the Committee recommended to the Board of Direction those modifications of the Act and its administration which, in the opinion of the Commitee members, would make the principle of the Act more workable and of wider application to the necessary public works of States, Counties, Municipalities, and other political sub-divisions or agencies.

Model Registration Law

For several years the Society has taken the leadership in formulating a law providing for the registration of engineers as eligible to practice. Several drafts have previously been devised, circulated, and discussed. This year, under the sponsorship of the Society's Committee on Registration, 23 representatives of collaborating societies came together and formulated what they have designated as the Model Law. This was "adopted and endorsed"

on July 4, 1932, by the Board of Direction, and has since become the standard projected for attainment whenever new registration laws are contemplated or revisions in existing laws proposed.

Engineers Council for Professional Development

Heartily, with others, the Society this year has joined in the setting up of a unit composed of three representatives from each of the following Societies: The American Society of Civil Engineers, the American Institute of Mining and Metallurgical Engineers, the American Society of Mechanical Engineers, the American Institute of Electrical Engineers, the American Institute of Chemical Engineers, the Society for the Promotion of Engineering Education, and the National Council of State Boards of Engineering Examiners. The objects are the formulation of successive criteria of technical, cultural, and professional training by which can be measured the advance of the young man toward professional status, and the determination of suitable methods of recognition of his attainment of that status. In effect, its objective is to create a practicable minimum definition of the profession.

Death of Dr. Hunt

In this eightieth year is to be especially recorded the death of Charles Warren Hunt. In 1892, he became Assistant Secretary and Librarian of the Society. In 1895, at the age of 37, he became Secretary, which position he held for 25 years; that is, until 1920, when he retired to become Secretary Emeritus. Dr. Hunt died July 23, 1932, at the age of 74 years.

MEETINGS OF THE BOARD OF DIRECTION

There have been five meetings of the Board of Direction during 1932:

January 18-19, New York, N. Y.
January 21, New York, N. Y.
May 9, New York, N. Y.
July 4-5, Yellowstone National Park, Wyoming.
October 3-4, Atlantic City, N. J.

There have been four meetings of the Executive Committee:

May 9, New York, N. Y.
July 4, Yellowstone National Park, Wyoming.
October 2, Atlantic City, N. J.
December 15, New York, N. Y.

MEMBERSHIP

The changes in membership are shown in the following table:

	Jan. 1, 1932			Jan. 1, 1933			Losses				Additions			Totals		
	Resident	Non-Resident	Total	Resident	Non-Resident	Total	Transfer	Resignation	Dropped	Died	Transfer	Election	Reinstatement	Loss	Gain	Increase
Honorary Members	5	13	18	5	14	19	...	...	.....	2	*3	.....	...	2	3	1
Members	1 022	4 848	5 870	992	4 829	5 821	3	34	44	115	†91	48	8	196	147	§49
Associate Members	1 081	5 256	6 337	1 090	5 241	6 331	91	73	148	22	‡107	208	13	334	328	§6
Juniors	553	2 282	2 835	549	2 407	2 956	107	47	¶194	7	....	472	4	355	476	121
Affiliates	42	83	125	37	78	115	0	4	5	1	....	.....	...	10	....	§10
Fellows	3	2	5	3	2	5	...	...	.....	...	...	.....	...	....	.....	...
Total	2 706	12 484	15 190	2 676	12 571	15 247	201	158	391	147	201	728	25	897	954	57

*3 Members.
†91 Associate Members.
‡107 Juniors.
§Decrease.
¶48 Juniors dropped on account of age limit.

New Members and Net Increase

The following table shows the new members and the net increase during the past ten years. The diagram on page 6 gives membership statistics for the same period:

	1923	1924	1925	1926	1927	1928	1929	1930	1931	1932
New Members*	733	715	795	1 072	1 139	1 244	1 066	1 139	1 055	753
Net Increase	320	262	111	721	755	820	508	574	531	57

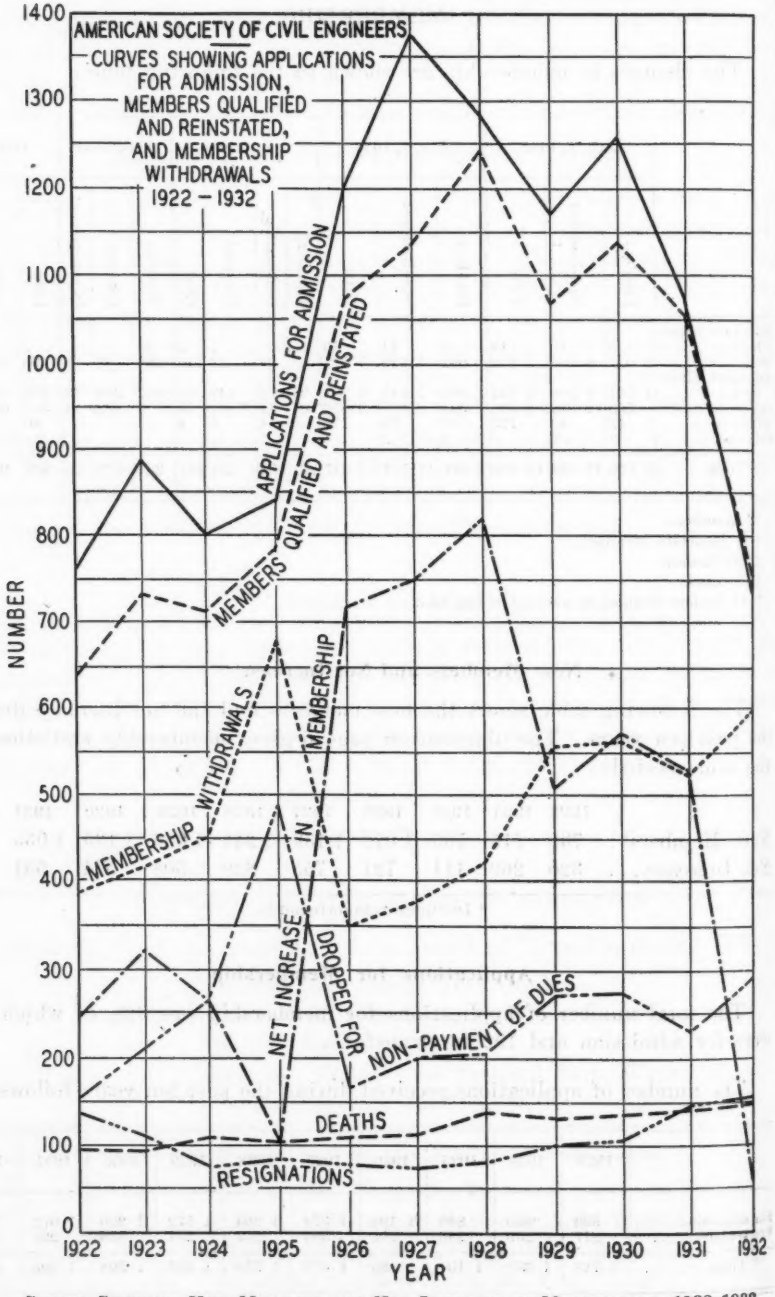
* Includes reinstatements.

Applications for Membership

The total number of applications for membership was 928, of which 736 were for admission and 192 for transfer.

The number of applications received during the past ten years follows:

	1923	1924	1925	1926	1927	1928	1929	1930	1931	1932
For admission	896	800	848	1 194	1 374	1 284	1 172	1 260	1 072	736
For transfer	217	239	256	292	304	274	271	338	224	192
Total	1 113	1 039	1 104	1 486	1 678	1 558	1 443	1 598	1 296	928



CURVES SHOWING NEW MEMBERS AND NET INCREASE IN MEMBERSHIP, 1922–1932.

DEATHS

The losses by death during the year number 147, and are as follows:

Past-Presidents (1):

John Ripley Freeman*

Honorary Members (1):

William Barclay Parsons

Members (115):

Frederick Wilhelm Albert
Truman Heminway Aldrich
Thomas Henry Alison
Percy Allan
Henry Clayton Allen
Christian Andersen
Basil Condon Battye
Frank George Baum
Bernard Arthur Behrend
Francis Wheelwright Belknap
Fred Milton Bisbee
Howard Edward Boardman
Willard Lewis Bowker
Samuel Dunlap Brady
John Severin Branne
Hugh Brodie
Walter Keith Brownell
Paul King Bunn
George Raymond Campbell
Frank Taylor Chambers
Elga Ross Chamblin
John Christian Valdemar Christensen
Oscar Claussen
Winfield Walker Conard
Herbert Chapin Daggett
Sverre Dahm
Louis Christian Datz
Arthur Lincoln Davis
Philip Hoffecker De Witt
Boyd Ehle
Arthur John Ela
George Ezra Ellis
Robert Lee Faris
John Moyer Farley
Henning Fernstrom
Hermann Fougner
Frank Hoyt Fowler
James Hillhouse Fuertes
John Peden Gardiner
Allan Vinal Garratt
Emerson Warren Grant
Edward Michael Graves
Robert Hall Gresham
John Wesley Hackney
Everett Addison Hadley
William John Hanna
Ferdinand Finney Harrington
Russell Theodore Hartman
Harrison Washburn Hayward
Arthur Rasco Hirst
Charles Warren Hunt
John Quintin Jamieson
Andrew Martin Jensen
Benjamin Oliver Johnson
William Henry Jones
Charles Henry Keefer
Warren Byron Keim
Jeremiah Joseph Kennedy
Morris Knowles
Edgar Ernest Larkins
Thomas Monahan Lavelle
John Walter Ledoux
Herbert Prescott Linnell
Henry Robertson Lordly
Alexander MacGlashan
Carl Arthur McClain
Arthur Pomeroy Melton
Manuel Maria Mendiola
George Nathan Merrill
William Selby Mitchell
Wesley Charnock Mock
Eusebius Joseph Molera
Frederick Haynes Newell
Stephen Henley Noyes
Kanji Okumura
Alfred Clarence Olney
Arthur Onderdonk
Samuel Henry Pitcher
Henry Tegmeyer Porter
George Addison Posey
Willis Ranney
Frederick George Ray
Tom Martin Reed
Frederick Jarrett Reinke
Abraham Moreau Reynolds
Carl Jonas Rhodin
Waldemar Spaulding Richmond
Earl Ivan Roberts
Edwin Henry Rogers
Albert Owen Rowse
Sumner Farnham Shaw
Frank Woodward Skinner
John Rodolph Slattery
Charles Churchill Small
Augustus Smith
Francis Pitt Smith
John Hammond Smith
Theodore Nelson Spencer
William Aiken Starrett
Vernon Christopher Suckow
Robert Swan Sumner
William Wolcott Tefft
William Love Thompson
Max Ernst Robert Toltz
William Francis Tye
William Leroy Ulrich
John Garret Van Horne
Otto von Geldern
Hans August Evald Conrad von Schon
William Thomas Walker
Joseph Doman Wardle
William Wheeler
George Monroe Wisner
Herbert Waldo York
Morris Alexander Zook

Associate Members (22):

Julian Wilmot Alger
George Castleman Breckinridge
Wendell Morris Butts
Francis Feher
Robert Maximilian Feustel
Arthur Fragoso de Lima Campos
Warren Adams Lyon
John Alexander McLean
John Bruce Mailey
Alfred Stowe Mirick
Joseph Lincoln Murphy
Thomas Hartman Olds
John Frederick Poland
Victor Jamison Roberts
Albert Henry Ross
Herbert Raymond Rowland
William Francis Sheehan
Paul Bertram Spencer
Joseph Edward Tempest
George Edward Wells
Frank Herman Winterer
Frank Dennett Yost

Affiliates (1):

Ernest Robinson Ackerman

Juniors (7):

Chelso Lino Arrigoni
James McCormick Brockway
Charles William Irons
Howard Rodgers Spelman
Robert Sigismond Sperling
Roy Joyce Vincent Thoren
William Rienze Visser

* Also an Honorary Member.

ENGINEERING SOCIETIES LIBRARY

The statistics which follow give comparative figures for 1931 and 1932 of the Engineering Societies Library:

Additions:	1931		1932	
Volumes (by gift)	2 106		2 452	
" (by purchase)	1 304	3 410	1 212	3 672
Pamphlets (by gift)	3 894		3 540	
" (by purchase)	251	4 145	322	3 862
Maps (by gift)	198		256	
" (by purchase)	10	208	17	273
Searches		94		31
Total additions		7 857		7 830
Permanent collection		144 093		144 648
Expenditures for books, periodicals, binding, supplies, and salaries (approximate)		$52 011		$48 197
The Library was used by		42 659		45 435
Including personal visits by		27 943		33 882
Volumes catalogued		4 477		4 179
Cards added to catalog		29 596		26 483
Total catalog cards, arranged under subject		450 939		477 422
Searches made		117		70
Translations made		242		138
Photoprints made		41 614		24 675
Number of persons securing photographs		4 585		3 041
Receipts for service		$17 031		$9 481
Members borrowing books		206		207

EMPLOYMENT SERVICE

The Employment Service has offices in New York, N. Y., Chicago, Ill., and San Francisco, Calif.

The number of men placed during 1932 has averaged about 48 per month. The following table shows the registrations and placements in the three offices:

Month	Men Registered				Men Placed			
	New York	Chicago	San Francisco	Total	New York	Chicago	San Francisco	Total
January	172	29	76	277	16	1	12	31
February	200	37	53	290	19	12	9	40
March	199	83	77	359	25	3	8	36
April	217	112	100	429	34	6	7	47
May	202	71	66	339	28	5	6	39
June	237	95	68	400	38	4	6	48
July	200	81	33	314	31	4	9	44
August	170	59	68	297	27	4	8	39
September	139	48	59	246	36	4	8	48
October	138	53	38	229	41	11	12	64
November	107	39	29	175	46	13	12	71
December	87	42	36	165	52	6	10	68
Total	2 068	749	703	3 520	393	73	107	573

PUBLICATIONS

One volume of *Transactions,* ten numbers of *Proceedings,* twelve numbers of *Civil Engineering,* and a Year Book have been published in 1932. The May number of *Proceedings* was issued in two parts: Part I consists of Papers and Discussions, and Part II contains the researches by Dr. Kyoji Suyehiro, entitled "Engineering Seismology: Notes on American Lectures."

Publication of Memoirs in *Proceedings* was discontinued with the September, 1930, number. Since that time approximately 270 memoirs have been published in pamphlet form many of which are included in *Transactions,* Vol. 95 (1931) and Vol. 96 (1932).

The stock of the various publications of the Society kept on hand for the convenience of members and others now amounts to 172 677 copies, the cost of which to the Society for paper and press work only has been $30 350.11.

The table (see page 10) shows the cost per page for text and illustrations in *Proceedings, Transactions,* and *Civil Engineering* for the past seventeen years.

The various topics developed in *Proceedings, Transactions,* and *Civil Engineering* during the year, and the number of pages devoted to each are as follows:

Subject	*Transactions,* pages	*Proceedings,* pages	*Civil Engineering,* pages
City and Regional Planning	28	.....	32
Contracts	.....	10	.....
Dams	188	269	57
Drainage and Irrigation	63	19	8
Earthquakes	.....	112	.....
Engineering Economics	.....	.....	30
Engineering Education	6	.....	.....
Engineering History	.....	46	4
Engineering Instruments	.....	.....	1
Engineering Profession	.....	.....	2
Forestation	.....	67	.....
Foundations	.....	.....	34
Geo-physics	.....	.....	5
Highway Engineering	35	97	19
Hydrology, Hydraulics	216	83	26
Industrial Operations	.....	.....	5
Materials of Engineering	44	34	13
Mathematics	114	.....	1
Meteorology	173	25	15
Power Plants	.....	30	.....
Power Transmission	.....	32	.....
Railways	.....	38	31
Refuse Disposal	.....	.....	4
Sanitation	.....	.....	2
Sewage Disposal	.....	.....	25
Soils	.....	99	.....
Structural Engineering	323	825	104
Surveying	52	69	26
Transportation	.....	.....	23
Tunnels	90	15	6
Valuation	.....	.....	3
Water Supply	.....	.....	12
Waterways	.....	38	10
Water-Works	73	42	.....
Letters to the Editor	.....	.....	96
Memoirs	256	.....	.....
Society Affairs	.....	27	120
Items of Interest	.....	.....	26
Local Sections	.....	.....	13
Student Chapters	.....	.....	9
Current Periodical Literature	.....	.....	36
Recent Books	.....	.....	6
Men and Positions Available	.....	.....	15
Changes in Membership Grades	.....	.....	16
News of Engineers	.....	.....	6
	1 641	1 977	841

Table Showing Number and Cost of Pages and Cost of Illustrations for *Transactions, Proceedings,* and *Civil Engineering.*

Year	Issues	Edition	Pages: Per volume	Pages: Total	Issues	Edition	Pages: Per volume	Pages: Total	Total pages	Total cost	Cost per page	Cost	Percentage of total cost.	Cost per page
	TRANSACTIONS				*PROCEEDINGS,* Part I*				*PROCEEDINGS* (Part I)* and *TRANSACTIONS*			Illustrations		
1916	1	8 400	2 301	19 330 000	10	8 350	2 892	24 140 000	43 470 000	35 645.65	0.00082	1 403.12	3.9	0.000032
1917	..	.....	.....	.....	10	8 550	3 432	29 350 000	29 350 000	28 608.18	0.00097	3 703.97	12.9	0.000126
1918	1	8 700	1 879	16 340 000	10	8 950	2 341	20 950 000	37 290 000	33 785.64	0.00091	1 192.20	3.5	0.000032
1919	1	9 000	1 775	15 980 000	8	9 100	2 096	19 075 000	35 055 000	32 082.69	0.00091	1 128.53	3.5	0.000032
1920	..	.....	.....	.....	10	10 142	2 014	20 440 000	20 440 000	23 446.34	0.00115	2 552.27	10.9	0.000125
1921	2	10 000 10 500	2 479 993	35 212 000	10	10 680	1 834	19 450 000	54 662 000	66 298.39	0.00121	2 034.72	3.1	0.000037
1922	1	10 900	1 826	19 900 000	10	11 100	2 740	30 400 000	50 300 000	56 200.00	0.00112	3 700.00	6.6	0.000073
1923	1	11 200	1 808	20 250 000	10	11 500	3 210	36 915 000	57 165 000	60 612.83	0.00106	4 809.88	7.9	0.000084
1924	1	11 500	1 515	17 440 000	10	11 750	2 612	30 700 000	48 140 000	47 573.72	0.00099	4 579.04	9.6	0.000095
1925	1	11 400	1 538	17 533 000	10	11 800	2 910	34 338 000	51 871 000	47 409.07	0.00091	6 359.19	13.4	0.000122
1926	..	12 100	1 786	21 610 000	10	12 200	3 046	37 161 000	58 771 000	53 473.54	0.00091	5 080.91	9.5	0.000087
1927	2	12 900 13 200	1 234 1 229	15 919 000 16 223 000	10	13 050	3 720	48 546 000	80 688 000	65 972.76	0.00080	9 279.49	14.0	0.000126
1928	1	14 000	1 860	26 040 000	10	14 200	4 016	57 027 000	83 067 000	61 587.98	0.00074	7 249.00	12.4	0.000087
1929	1	14 500	2 030	29 435 000	10	14 540	3 566	51 850 000	81 285 000	58 388.39	0.00072	4 011.95	6.9	0.000049
1930	1	15 100	1 794	27 090 000	10	15 170	2 936	44 540 000	71 630 000	52 881.95	0.00074	4 189.56	7.9	0.000058
1931	1	15 500	1 720	26 660 000	10	15 820	1 968	31 133 000	57 793 000	42 342.33	0.00073	3 680.95	8.7	0.000064
1932	1	15 800	1 688	26 670 000	10	15 700	2 160	34 010 000	60 680 000	37 263.66	0.00061	4 830.47	13.0	0.000080
					CIVIL ENGINEERING									
1930	..	.....	.....	.....	3	16 230	248	4 026 000	4 026 000	8 969.66	0.00223	995.06	10.1	0.000247
1931	..	.....	.....	.....	12	16 275	1 264	20 572 000	20 572 000	41 012.75	0.00199	6 271.40	15.3	0.000305
1932	..	.....	.....	.....	12	16 020	1 048	16 786 000	16 786 000	32 775 39	0.00195	5 112.68	15.6	0.000305

* Includes Part III, May 1928, 288 pp.; Part 2, May 1932, 112 pp.

Summary of Publications for 1932

	Issues	Average edition	Total pages	Plates	Cuts
Proceedings (monthly numbers)	10	15 760	2 048	..	682
Proceedings, Part II (monthly number)	1	16 500	110	..	65
Civil Engineering (monthly numbers)	12	16 000	1 000	..	946
Transactions, Vol. 96	1	15 800	1 686	..	468
Year Book	1	16 000	510	1	3
Total	25	....	5 354	1	2 164

The gross cost of publications, as determined by the bills actually paid during the year, and inclusive of salaries, has been:

Technical Publications	$122 866.67
General Publications	6 898.04
Total	$129 764.71

READING ROOM OF THE SOCIETY

The attendance at the Reading Room during the year was 4 274.

Two hundred and sixty-seven periodicals are regularly received. Included in this number are many foreign periodicals, also a number of literary magazines and several daily newspapers.

MEETINGS

Five meetings of 7 sessions were held during the year as follows: At the Annual Meeting, at New York, N. Y., 1 (1 session); at the Annual Convention, at Yellowstone National Park, Wyoming, 1 (2 sessions); at the Fall Meeting, at Atlantic City, N. J., 1 (2 sessions); and 2 regular meetings of the Society held in Engineering Societies Building, New York, N. Y.

At the Quarterly Meetings of the Society, there were presented 3 formal papers, 1 Symposium, 3 Reports of Special Committees of the Society, and 9 Addresses. Most of these papers have been published in *Proceedings* and *Civil Engineering* as noted subsequently. A Student Chapter Conference was held at Atlantic City, N. J.

During the year the Second Progress Report of Sub-Committee No. 31, Committee on Steel, of the Structural Division on Wind-Bracing in Steel Buildings (presented at the Annual Meeting in New York), and Standard Symbols and Glossary for Hydraulics and Irrigation, compiled by the Special Committee on Irrigation Hydraulics, were published in February and May, 1932, *Proceedings,* respectively. The May *Proceedings* was published in two parts: Part I was the regular monthly *Proceedings,* and Part II was devoted to a series of addresses by the late Dr. Kyoji Suyehiro, entitled "Engineering Seismology: Notes on American Lectures." There have also been

published in *Proceedings* 29 papers and 1 Symposium. These papers include four of the series being published jointly by the Port of New York Authority and the Society, which appear in August, October, and December 1932, *Proceedings*, respectively.

The number of members and others taking part in the preparation and discussion of these papers, discussions, Symposium, and reports published in *Proceedings* was 212, and 123 discussions on papers published in 1931 *Proceedings* were also included in *Proceedings* for 1932.

The total attendance at the meetings of the Society was approximately 2 525. The registered attendance at the Annual Meeting was, 1 776; at the Annual Convention, 249; and at the Fall Meeting, 493.

The dates of the meetings of the Society during the year, together with the titles of papers (abstracts), addresses, etc., presented thereat, are as follows:

January 20, 1932 (One Session), Reports of Special Committees on Earths and Foundations; Irrigation Hydraulics; and Salaries.

July 6, 1932 (Two Sessions)[1], "Some Reflections Regarding Engineering Practice,"[2] Address by Herbert S. Crocker, President, Am. Soc. C. E.; "The National Parks and Their Improvement," by Frank A. Kittredge, M. Am. Soc. C. E.; "The Rise and Fall of the Public Domain," by Herman Stabler, M. Am. Soc. C. E.; and "Forest and Stream Flow,"[3] by W. G. Hoyt, M. Am. Soc. C. E., and H. C. Troxell, Assoc. M. Am. Soc. C. E.; and Geology of Yellowstone Park, an Address by Dr. Richard M. Field.

October 5, 1932 (Two Sessions),[4] Symposium on "The Regional Plan of the Philadelphia Tri-State District."

The papers, etc., published in *Proceedings* during the year were as follows:

January, 1932, "Wind-Bracing Connection Efficiency," by U. T. Berg, Assoc. M. Am. Soc. C. E.; "Public Supervision of Dams:" A Symposium; "Stereo-Topographic Mapping," by C. H. Birdseye, M. Am. Soc. C. E.; and "Determination of Principal Stresses in Buttresses and Gravity Dams," by W. H. Holmes, Assoc. M. Am. Soc. C. E.

February, 1932, "Economic Proportions and Weights of Modern Highway Cantilever Bridges," by J. A. L. Waddell, M. Am. Soc. C. E.; "Design Characteristics of the Reading Overbuild Transmission Line," by Frederick W. Deck, Assoc. M. Am. Soc. C. E.; Stresses in Reinforced Concrete Due to Volume Changes," by C. P. Vetter, Assoc. M. Am. Soc. C. E.; and Second Progress Report of Sub-Committee No. 31, Committee on Steel, of the Structural Division on Wind-Bracing in Steel Buildings.

March, 1932, "Suspension Bridges Under the Action of Lateral Forces," by Leon S. Moisseiff, M. Am. Soc. C. E., and Frederick Lienhard, Esq.; "Construction Methods on the Welland Ship Canal," by E. G. Cameron, Esq.; and "Stresses in Inclined Arches of Multiple-Arch Dams," by George E. Goodall and Ivan M. Nelidov, Associate Members, Am. Soc. C. E.

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[1] *Civil Engineering*, September, 1932, pp. 533 and 541.
[2] *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 1380.
[3] *Proceedings*, Am. Soc. C. E., August, 1932, p. 1037.
[4] *Civil Engineering*, December, 1932, p. 721.

April, 1932, "The Compensated Arch Dam," by A. V. Karpov, M. Am. Soc. C. E.; and "Development of Hydro-Electric Power as an Aid to Irrigation," by Charles C. Cragin, M. Am. Soc. C. E.

May, 1932, Part 2, "Engineering Seismology: Notes on American Lec- Navigable Fairway," by Lynne J. Bevan, M. Am. Soc. C. E.; "Application of Duration Curves to Hydro-Electric Studies," by G. H. Hickox and G. O. Wessenauer, Juniors, Am. Soc. C. E.; and Standard Symbols and Glossary for Hydraulics and Irrigation: Compiled by the Special Committee on Irrigation Hydraulics.

May, 1932, Part 2, "Engineering Seismology: Notes on American Lectures," by Dr. Kyoji Suyehiro.

August, 1932, "George Washington Bridge: General Conception and Development of Design," by O. H. Ammann, M. Am. Soc. C. E.; and "Forests and Stream Flow," by W. G. Hoyt, M. Am. Soc. C. E., and H. C. Troxell, Assoc. M. Am. Soc. C. E.

September, 1932, "Tests of Riveted and Welded Steel Columns," by Willis A. Slater and M. O. Fuller, Members, Am. Soc. C. E.; "The Martinez-Benicia Bridge," by W. H. Kirkbride, M. Am. Soc. C. E.; and "Pre-Qualification of Contractors," by C. J. Tilden, M. Am. Soc. C. E.

October, 1932, "Tests for Hydraulic-Fill Dams," by Harry H. Hatch, M. Am. Soc. C. E.; "George Washington Bridge: Organization, Construction Procedure, and Contract Provisions," by Edward W. Stearns, M. Am. Soc. C. E.; and "Foundation Treatment at Rodriguez Dam," by Charles P. Williams, M. Am. Soc. C. E.

November, 1932, "A History of the Development of Wooden Bridges," by Robert Fletcher and J. P. Snow, Members, Am. Soc. C. E.; "Distribution of Shear in Welded Connections," by Henry W. Troelsch, M. Am. Soc. C. E.; "Work of Rivets in Riveted Joints," by A. Hrennikoff, Esq.; and "Study of Stilling-Basin Design," by C. Maxwell Stanley, Jun. Am. Soc. C. E.

December, 1932, "A Problem of Soil in Transportation in the Colorado River," by S. L. Rothery, M. Am. Soc. C. E.; "George Washington Bridge: Design of Superstructure," by Allston Dana and Aksel Andersen, Members, Am. Soc. C. E., and G. M. Rapp, Assoc. M. Am. Soc. C. E.; "George Washington Bridge: Design of the Steel Towers," by Leon S. Moisseiff, M. Am. Soc. C. E.; and "Model Law for Motion of Salt Water Through Fresh," by Morrough P. O'Brien and John Cherno, Associate Members, Am. Soc. C. E.

MEDALS, PRIZES, AND AWARDS

The award of Medals and Prizes for the year ending July, 1932, was as follows:

The J. James R. Croes Medal to David L. Yarnell and Floyd A. Nagler, Members, Am. Soc. C. E., for their paper entitled "Effect of Turbulence on the Registration of Current Meters."

The Thomas Fitch Rowland Prize to Clifford Allen Betts, M. Am. Soc. C. E., for his paper entitled "Completion of Moffat Tunnel of Colorado."

The James Laurie Prize to Earl I. Brown, M. Am. Soc. C. E., for his paper entitled "The Chesapeake and Delaware Canal."

The Arthur M. Wellington Prize to Fred Lavis, M. Am. Soc. C. E., for his paper entitled "Highways as Elements of Transportation."

The Collingwood Prize for Juniors to Arthur R. C. Markl, Jun. Am. Soc. C. E.,[5] for his paper entitled "The Shannon Power Development in the Irish Free State."

Two Traveling Scholarships were awarded, from the Freeman Fund, for the year 1932-1933. The recipients of these scholarships are Herbert H. Wheaton, Assoc. M. Am. Soc. C. E., and Donald P. Barnes, Jun. Am. Soc. C. E.

The second award of the Alfred Noble Prize was made to Frank M. Starr, for his paper "Equivalent Circuits," published in the *Transactions* of the American Institute of Electrical Engineers, Vol. 51, No. 2, June, 1932.

LOCAL SECTIONS

There are at present 56 Local Sections. The Chattanooga Section, the Ithaca Section, and the Indiana Section approved by the Board of Direction on January 18, January 21, and July 4, 1932, respectively, have been added during the year.

TECHNICAL DIVISIONS

During the year there were eighteen sessions of Technical Divisions held in connection with Society meetings. Four of the Division programs were double sessions and four were joint sessions of two Divisions.

At the 1932 Annual Meeting the General Meeting on Wednesday afternoon was held under the auspices of the Engineering-Economics and Finance Division.

In general, the Technical Division sessions were well attended and served to broaden the scope and enlarge the interest in the Society meetings. The policy of publishing abstracts in *Civil Engineering* of papers, discussions, and reports presented at the Division sessions, has been continued during 1932.

City Planning Division

January 21, 1932,[6] "Location of Underground Facilities," by Arthur W. Consoer, M. Am. Soc. C. E.; and "Street Names and Numbers," by J. P. Schwada, Assoc. M. Am. Soc. C. E.

October 6, 1932[7] (Joint Session with Surveying and Mapping Division), "The City Plan—A Summarization of What It Should Include," by E. P. Arneson, M. Am. Soc. C. E.; and "Types of Surveys Used in Regional and City Planning and the Reasons for Using Them," by Russell V. Black, M. Am. Soc. C. E.

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[5] Now Associate Member.

[6] *Civil Engineering*, March, 1932, pp. 176, 186.

[7] *Loc. cit.*, December, 1932, pp. 750, 753.

Construction Division

January 21, 1932[8] (Joint Session with Structural Division), Report of Sub-Committee on Wind-Bracing on Tall Buildings;[9] "Building Foundations," by C. S. Proctor, M. Am. Soc. C. E.; "Subway Foundations," by Lazarus White, M. Am. Soc. C. E.; and "Bridge Foundations," by J. J. Yates, M. Am. Soc. C. E.

July 7, 1932,[10] "Construction Equipment of Hoover Dam," by Norman Gallison, Esq.

Engineering-Economics and Finance Division

January 20, 1932,[11] "The Vertical Clearance Required at Bridges Below Seaports," by C. E. Grunsky, Past-President, Am. Soc. C. E.; "Legal Phases of the Subject of Bridge Clearances," by Walter H. Wheeler, M. Am. Soc. C. E.; "Present Status of Valuation Procedure," by E. B. Black, M. Am. Soc. C. E.; and "Long Range Planning of Construction," by D. H. Sawyer, M. Am. Soc. C. E.

October 7, 1932,[12] "The Relation of Investment in Capital Facilities and Construction to the Economic Scheme," by John P. Hogan, M. Am. Soc. C. E.; "Public Works Financing," by Malcolm Pirnie, M. Am. Soc. C. E.; and "Combining Compulsory Corporate Reserves and Public Works Construction for National Economic Stability," by Frederick A. McDonald, M. Am. Soc. C. E.

Highway Division

January 20, 1932,[13] "New Jersey Approaches to the George Washington Bridge," by J. L. Bauer, M. Am. Soc. C. E.; and "Unemployment Relief Activities in Twin Cities," by William M. Carey, M. Am. Soc. C. E.

July 7, 1932[14] (Two Joint Sessions with the Structural Division), "Western Highway Bridge Practice," by C. B. McCullough, M. Am. Soc. C. E.; "Roads in the National Parks," by L. I. Hewes, M. Am. Soc. C. E.; "Steel Pile Foundations for Highway Bridges," by J. G. Mason, M. Am. Soc. C. E., and Alfred L. Ogle, Assoc. M. Am. Soc. C. E.; and "New Types of Reinforced Concrete Highway Bridges," by J. W. Beretta, Esq.

October 7, 1932,[15] "Traffic Control from the Standpoint of the State Highway System," by J. L. Bauer, M. Am. Soc. C. E.; "Traffic Control from the Standpoint of City Traffic," by Burton W. Marsh, M. Am. Soc. C. E.; and "Can the Highway Investment Be Justified on Economic Grounds?" by T. R. Agg, M. Am. Soc. C. E.

Irrigation Division

July 7, 1932[16] (Two Joint Sessions with Power Division), "The Development of the Columbia River," by Thomas M. Robins, Esq.; "Effect of Uplift

[8] *Civil Engineering*, March, 1932, pp. 137 *et seq.*
[9] *Proceedings*, Am. Soc. C. E., February, 1932, p. 213.
[10] *Civil Engineering*, September, 1932, p. 573.
[11] *Loc. cit.*, March, 1932, pp. 147 *et seq.*
[12] *Loc. cit.*, December, 1932, pp. 731 *et seq.*
[13] *Loc. cit.*, March, 1932, pp. 158, 160.
[14] *Loc. cit.*, September, 1932, p. 537, *et seq.*
[15] *Loc. cit.*, December, 1932, pp. 770, 774, 761.
[16] *Loc. cit.*, September, 1932, p. 546 *et seq.*

on Stability of Straight Gravity Dams," by D. C. Henny, M. Am. Soc. C. E.; "Duty of Water in Terms of Canal Capacity," by E. B. Debler, M. Am. Soc. C. E.; and "Tests for Hydraulic-Fill Dams,"[17] by Harry H. Hatch, M. Am. Soc. C. E.

Power Division

January 21, 1932,[18] "Federal Power Commission's Field of Work," by Dr. George Otis Smith.

Sanitary Engineering Division

January 21, 1932,[19] Report of Committee on Progress and Status of the Art of Sanitary Engineering; "The Activated Sludge Process of Sewage Treatment," by Wellington Donaldson, M. Am. Soc. C. E.; and Report of Committee on Sludge Digestion.

October 6, 1932,[20] "The Washington, D. C., Refuse Problem," by Harrison P. Eddy, Jr., M. Am. Soc. C. E.; "The Joint Trunk Sewer for the Hackensack District," by George W. Fuller, M. Am. Soc. C. E.; and "Sewage Disposal of the New Jersey Coast Resorts," by Chester G. Wigley, M. Am. Soc. C. E.

October 7, 1932,[21] "Problems of Water Supply, Sewerage, and Industrial Waste Disposal in Delaware County, Pennsylvania," by H. M. Freeburn, Assoc. M. Am. Soc. C. E.; and "Ground-Water Resources of New Jersey," by Howard T. Critchlow, M. Am. Soc. C. E.

Surveying and Mapping Division

January 21, 1932,[22] "Geodetic Control for North Carolina Highway, Its Advantages and Its Probable Effect on Engineering Practice in the State," by George W. Syme, M. Am. Soc. C. E.; and "The Place of Surveying in Engineering Education," by James K. Finch, M. Am. Soc. C. E.

MEMBERSHIP OF TECHNICAL DIVISIONS

There are now 14 794 members enrolled in the Technical Divisions as follows:

City Planning	1 553
Construction	2 667
Engineering-Economics and Finance	405
Highway	2 182
Irrigation	939
Power	875
Sanitary Engineering	1 655
Structural	2 790
Surveying and Mapping	870
Waterways	858

[17] *Proceedings*, Am. Soc. C. E., October, 1932, p. 1301.
[18] *Civil Engineering*, March, 1932, p. 153.
[19] *Loc. cit.*, pp. 164 *et seq.*
[20] *Loc. cit.*, December, 1932, pp. 746, 763, 743.
[21] *Loc. cit.*, December, 1932, pp. 756, 774.
[22] *Loc. cit.*, March, 1932, pp. 180, 183.

STUDENT CHAPTERS

There are at present 103 Student Chapters. The following were organized during 1932:

Louisiana State University
North Dakota State College
Rhode Island State College
University of Delaware

The reports of the Secretary and Treasurer are appended.

By order of the Board of Direction,

GEORGE T. SEABURY,
Secretary.

January 16, 1933.

REPORT OF SECRETARY FOR THE

TO THE BOARD OF DIRECTION OF THE

GENTLEMEN:—I have the honor to present a statement of Receipts and
There is also appended a general Balance Sheet showing the condition of the

RECEIPTS

Cash on hand January 1, 1932			$28 140.31*
Entrance Fees		$13 300.00	
Current Dues		158 296.19	
Past Dues		8 558.10	
Advance Dues		40 248.64	
Sale of Publications		9 475.23	
Binding for Members		7 506.00	
Badges		3 570.50	
Certificates of Membership		418.50	
Annual Meeting		3 469.00	
Interest on Deposits		274.61	
Interest on Investments		2 245.79	
Interest Accrued		108.00	
Postage		402.26	
Advertising		22 817.14	
Miscellaneous		723.20	
Special Publications		2 680.00	
Sale of Securities		28 272.56	
Securities called (4 000 Chicago Tax Warrants)		4 025.19	
Income from 57th St. Property:			
Credited to General Receipts	$43 000		
Credited to 57th St. Property Funds	9 500	52 500.00	
From Engineering Foundation in credit to:			
Special Committee on Earths and Foundations	$2 000		
Special Committee on Steel Column Research	2 000		
Special Committee on Concrete and Reinforced Concrete Arches	1 500	5 500.00	
The 57th St. Property Fund:			
Interest on Securities		2 692.65	
Interest Accrued		194.85	
Loan from Chase National Bank	$10 000		
Less Discount	150	9 850.00	
The Freeman Fund:			
Income and Expense		1 678.29	
The Alfred Noble Fund:			
Sale of Securities		3 462.15	
Interest on Invested Funds		794.73	
Interest on not Released Funds		4.28	
City Planning Division:			
Dues		20.00	
Interest		17.64	
Surveying and Mapping Division:			
Dues		15.00	
Power Division:			
Interest		76.50	
Rudolph Hering Medal Fund:			
Interest		20.16	
Merritt H. Smith Memorial:			
Interest		39.71	383 256.87
			$411 397.18

* For itemized statement, see page 20.

YEAR ENDING DECEMBER 31, 1932

AMERICAN SOCIETY OF CIVIL ENGINEERS,

Disbursements for the fiscal year of the Society, ending December 31, 1932. affairs of the Society.

Respectfully submitted,

GEORGE T. SEABURY,
Secretary.

DISBURSEMENTS

Salaries of Officers	$21 215.00
Retirement Allowances	4 454.48
Clerical Help	69 069.99
Traveling Allowance of Officers	15 020.14
Rent	13 208.49
Telephone	2 040.36
General Publications	6 898.04
General Printing	1 874.76
Postage	5 199.53
Binding	4 299.39
Badges	2 342.08
Certificates	310.77
Annual Prizes	346.53
Office Supplies	2 754.80
Furniture and Office Equipment	1 320.75
Current Business	5 315.00
Interest on Mortgage	1 500.00
Insurance	265.61
Reading Room	461.71
Miscellaneous	868.09
Employment Service	4 639.92
Library	8 743.56
American Standards Association	507.00
Local Sections	7 327.00
Technical Publications	122 866.67
Meetings	8 592.59
Technical Divisions	4 480.92
Technical Committees	9 125.21
Administrative Committees	1 068.94
Professional Committees	3 970.21
American Engineering Council	12 911.48
Construction League	300.00
The 57th St. Property Fund:	
Purchase of Securities	13 575.00
Interest Accrued	194.85
Loan from Chase National Bank	10 000.00
The Freeman Fund:	
Income and Expense	2 276.50
The Alfred Noble Fund:	
Income and Expense	1 536.50
	$370 881.87
Cash on hand December 31, 1932	40 515.31*
	$411 397.18

ITEMIZED STATEMENT OF CASH ON HAND JANUARY 1, 1932

Society Funds		$4 640.93
57th Street Property Fund		13 306.17
Power Division		2 156.50
City Planning Division		493.77
Surveying and Mapping Division		75.00
Special Committee on Stresses in Railroad Track	$360.28	
Special Committee on Earths and Foundations	2 424.17	2 784.45
The Freeman Fund:		
Income and Expense	$638.08	
Principal	67.20	705.28
In Escrow, E. W. L.		393.02
The Alfred Noble Fund:		
Uninvested	$127.24	
Not Released	55.00	
Income and Expense	1 581.47	1 763.71
Merritt H. Smith Memorial Fund:		
Principal	$1 076.50	
Income and Expense	43.78	1 120.28
Rudolph Hering Medal Fund:		
Principal	$455.97	
Income and Expense	39.20	495.17
Library and Prize Fund		206.03
		$28 140.31

ITEMIZED STATEMENT OF CASH ON HAND DECEMBER 31, 1932

Society Funds in Chase National Bank 23d Street	$14 248.97	
Society Funds in Chase National Bank 41st Street	500.00	
Petty Cash (in hands of Secretary)	5 000.00	$19 748.97
The 57th St. Property Fund		11 923.82
Power Division		2 233.00
City Planning Division		531.41
Surveying and Mapping Division		90.00
Special Committee on Stresses in Railroad Track		60.28
Special Committee on Steel Column Research		333.70
Special Committee on Earths and Foundations		2 351.59
In Escrow, E. W. L.		393.02
The Freeman Fund:		
Income and Expense	$39.87	
Principal	67.20	107.07
The Alfred Noble Fund:		
Uninvested	$189.39	
Not Released	55.00	
Income and Expense	822.74	1 067.13
The Merritt H. Smith Memorial Fund:		
Principal	$1 076.50	
Income and Expense	83.49	1 159.99
The Rudolph Hering Medal Fund:		
Principal	$455.97	
Income and Expense	59.36	515.33
		$40 515.31

REPORT OF THE TREASURER OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS FOR THE YEAR ENDING DECEMBER 31, 1932

In compliance with the provisions of the Constitution, I have the honor to present the following report:

Cash on hand January 1, 1932		$28 140.31
Receipts		
From Current Sources, January 1 to December 31, 1932	$300 238.52	
Rent from 57th Street Property	52 500.00	
From Sale of Securities	28 272.56	
Interest on Investments	2 245.79	383 256.87
Disbursements		
Payment of Bills by Audited Vouchers, January 1 to December 31, 1932	$370 881.87	
Cash on hand December 31, 1932	40 515.31	
	$411 397.18	$411 397.18

Respectfully submitted,

OTIS E. HOVEY,
Treasurer.

BALANCE SHEET

ACCOMPANYING THE REPORT

ASSETS

Current Assets:			
Cash:			
In banks and on hand	$1 671.97		
On deposit with U. S. Post Office	200.00	$1 871.97	
Investment in City of Chicago tax notes (due September 15, 1930), at cost, and accrued interest		7 297.79	
Accounts receivable:			
Members	$63 728.81		
Non-members	4 184.94		
	$67 913.75		
Less, allowance for doubtful accounts	$40 690.64	27 223.11	
Inventory of publications on hand, at cost or less		10 969.03	
Prepaid insurance premium		14.00	
		$47 375.90	
Real Estate:			
Interest in real estate and other assets of United Engineering Trustees, Inc., exclusive of trust funds	$493 352.60		
218-220 West 57th Street, New York, N. Y., at estimated value, less depreciation	607 320.04	1 100 672.64	
Furniture and office equipment, less reserve for depreciation		4 666.73	
Library:			
Cash expended for books, etc.	$22 122.22		
Donations (estimated)	72 310.83	94 433.05	$1 247 148.32
Fund Investments:			
Library and Prize Funds, etc.:			
Marketable securities, at cost (market value $5 580)	$7 690.50		
Accrued interest	85.00		
Uninvested cash	18 077.00	25 852.50	
The Fifty-seventh Street Property Fund:			
Marketable securities, at cost (market value $43 730)	$62 654.24		
Accrued interest	1 090.11		
Uninvested cash	11 923.82	75 668.17	
The Freeman Fund:			
Marketable securities, at cost (market value $17 650)	$26 018.93		
Uninvested cash	67.20	26 086.13	
Alfred Noble Fund:			
Marketable securities, at cost (market value $10 507)	$14 856.25		
Accrued interest	242.50		
Uninvested cash	189.39	15 288.14	
Merritt Haviland Smith Memorial Fund		1 159.99	
Rudolph Hering Medal Fund		515.33	144 570.26
Cash in banks, representing unexpended balances of cash received for special purposes			6 910.61
			$1 398 629.19

We have examined the accounts of the AMERICAN SOCIETY OF CIVIL ENGINEERS to the reasonableness of the indicated property and library estimated valuations, we of the Society at that date.

NEW YORK, January 12, 1933.

DECEMBER 31, 1932

OF THE SECRETARY

LIABILITIES AND FUNDS

Mortgage payable, due February 1, 1935...	$30 000.00		
Interest accrued on mortgage	625.00		
1933 membership dues paid in advance.....	40 248.64		
Other membership credits, etc.....	2 337.42	$73 211.06	
Current fund surplus, including amount arising from revaluation of real estate.....		1 173 937.26	$1 247 148.32
Funds:			
Library and Prize Funds:			
Herbert Stewart Library Fund.....	$2 000.00		
Joseph G. Swift Library Fund	1 000.00		
Compounded Dues Fund	15 480.00		
Norman Medal Fund	1 000.00		
Rowland Prize Fund	1 222.50		
Collingwood Prize Fund	1 000.00		
Arthur M. Wellington Prize Fund....	2 150.00		
Hiram F. Mills Legacy.....	2 000.00	$25 852.50	
The Fifty-seventh Street Property Fund.....		75 668.17	
The Freeman Fund		26 086.13	
Alfred Noble Fund, with accrued interest.....		15 288.14	
Merritt Haviland Smith Memorial Fund.....		1 159.99	
Rudolph Hering Medal Fund.....		515.33	144 570.26
Unexpended balances of cash received for special purposes:			
Special Committee on Stresses in Railroad Track....		$60.28	
Special Committee on Earths and Foundations.....		2 351.59	
Power Division		2 233.00	
City Planning Division		531.41	
Surveying and Mapping Division.....		90.00	
Freeman Fund income		39.87	
Alfred Noble Fund		877.74	
E. W. L. (in escrow)		393.02	
Special Committee on Steel Column Research.....		333.70	6 910.61
			$1 398 629.19

as at December 31, 1932, and upon the basis of carrying securities at cost and subject certify that, in our opinion, the above balance sheet sets forth the financial condition

LYBRAND, ROSS BROS. & MONTGOMERY,
Accountants and Auditors.

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer*, and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibility. It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from March 15, 1933.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct important work	35 years	12 years*	5 years of important work
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years of important work
Fellow	Contributor to the permanent funds of the Society			

*Graduation from a school of engineering of recognized reputation is equivalent to 4 years of active practice.

†Membership ceases at age of 33 unless transferred to higher grade.

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

FOR ADMISSION

BAILEY, WILLIAM ATTERBURY, Berkeley, Cal. (Age 27.) Refers to C. E. Andrew, J. S. Bates, C. R. Harding, F. R. Muhs, D. R. Warren.

BARKSDALE, HENRY COMPTON, Trenton, N. J. (Age 34.) Asst. Engr., New Jersey State Water Policy Comm. Refers to B. A. Bennett, J. N. Brooks, S. Burnley, H. T. Critchlow, O. W. Hartwell, O. Lauterhahn, A. Richards, H. A. Yancey.

BURN, PHILIP HAVILAND, Brooklyn, N. Y. (Age 35.) Refers to J. E. Boatrite, A. H. Bull, C. E. Conover, I. Jacobs, R. Ridgway, H. D. Winsor.

COLE, EDWARD SHAW, Upper Montclair, N. J. (Age 25.) Refers to C. M. Allen, J. W. Alvord, E. D. Case, E. S. Cole, W. A. Conley, R. R. Marsden, W. C. Parmley, E. K. Wilson.

DIECK, WILLIAM HENRY, New York City. (Age 34.) Asst. Engr., United Elec. Light & Power Co. Refers to J. L. Bogart, F. A. Ericson, T. R. Galloway, H. P. Hammond, E. J. Squire.

FOLGNER, ARTHUR MAXWELL, New York City. (Age 26.) Jun. Engr., Board of Transporation. Refers to B. J. Ahearn, H. B. Gates, R. P. Gustin, E. G. Haines, B. Kipp, A. F. Simpson.

HAINES, DORSEY DONALD, Lawrence, Kans. (Age 29.) Instructor in Civ. Eng., Univ. of Kansas. Refers to W. E. Baldry, J. L. Barron, E. Boyce, J. O. Jones, W. C. McNown, H. A. Rice.

KEMP, KENNETH SMITH, Endicott, N. Y. (Age 25.) Refers to E. R. Cary, L. W. Clark, T. R. Lawson, W. W. Rousseau, H. O. Sharp.

KING, FOREST GOLDIE, Seattle, Wash. (Age 43.) Civ. and Cons. Engr. Refers to G. Kimbrell, O. Laurgaard, T. G. McCrory, B. S. Morrow, W. C. Morse, L. V. Murrow, F. M. Randlett, O. E. Stanley.

MANLEY, RICHARD GEORGE, Upland, Cal. (Age 42.) City Service Mgr. Refers to J. C. Albers, F. C. Froehde, G. S. Hinckley, E. A. Ingham, E. Q. Sullivan.

NIVEN, STANLEY, White Plains, N. Y. (Age 42.) Asst. Engr. with W. W. Young, Cons. Engr. for Westchester County San. Sewer Comm. Refers to A. M. Brosius, F. J. Laverty, L. G. Rice, D. J. Shaw, A. M. Wyman, F. C. Zeigler.

NOSER, JOE EDWARD, Mertzon, Tex. (Age 25.) With Earl Beavers, San Angelo, Tex. Refers to J. A. Focht, T. U. Taylor.

REDICK, FLOYD CARL, Columbus, Ohio. (Age 27.) Refers to E. F. Coddington, C. T. Morris, J. C. Prior, C. E. Sherman, R. C. Sloane.

RIDENOUR, CLYDE OTMER, Cincinnati, Ohio. (Age 26.) With Columbia Eng. and Management Corporation. Refers to I. T. Heller, C. T. Morris.

RUDNICK, STANLEY SHERMAN, Brooklyn, N. Y. (Age 21.) Refers to C. B. Breed, W. M. Fife.

SCHROEDER, WALTER NICHOLAS, Springfield, Ill. (Age 51.) Asst. Bridge Engr., Illinois Highway Dept. Refers to R. R. Benedict, G. F. Burch, E. D. Dryfoose, A. H. Fuller, F. T. Sheets, W. M. Wilson.

SULLIVAN, EUGENE BINGHAM, East Hartford, Conn. (Age 25.) Refers to R. H. Buck, C. E. Sherman.

TAYLOR, BYRON KIZER, Eugene, Ore. (Age 28.) Asst. Engr. (Civ.), Water Board. Refers to A. L. Alin, G. H. Canfield, C. A. Mockmore, H. S. Rogers, W. L. Sharp, J. C. Stevens.

TETER, CLYDE WILLIAM, Brooklyn, N. Y. (Age 23.) Refers to R. E. Goodwin, F. O. X. McLoughlin, J. S. Peck, J. C. Rathbun.

THOMPSON, JOHN CAVETT, Albuquerque, N. Mex. (Age 23.) Tracer, Middle Rio Grande Conservancy Dist. Refers to J. H. Dorroh, W. B. Ream, H. L. Thompson.

WICKES, ROBERT LINDSAY, New York City. (Age 23.) Engr., George J. Atwell Foundation Co. Refers to W. J. Barney, T. Crane, A. Durant, C. S. Farnham, R. H. Suttie, C. J. Tilden, J. C. Tracy.

WILSON, MILTON THEURER, San Francisco, Cal. (Age 34.) Asst. Engr., Water Resources Branch, U. S. Geological Survey. Refers to J. G. Bastow, R. C. Briggs, N. C. Grover, O. W. Israelsen, H. D. McGlashan, C. G. Paulsen, A. B. Purton, E. Schaub, R. B. West.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

ASH, CHARLES, Assoc. M., Lima, Ohio. (Elected Nov. 12, 1928.) (Age 50.) Div. Engr., Ohio State Highway Dept. Refers to V. A. Eberly, D. R. Keith, A. E. Morgan, G. A. Reese, G. F. Schlesinger, R. H. Smith.

De MERSE, FRANCIS FELIX, Assoc. M., Phoenix, Ariz. (Elected Aug. 18, 1930.) (Age 37.) Asst. to Dist. Engr., Maricopa County Municipal Water Conservation Dist. No. 1. Refers to W. H. Becker, C. E. Griggs, A. F. Harter, W. W. Lane, J. B. Lippincott, T. Maddock, E. V. Miller.

GRANTZ, WALTER ANTON HENRY, Assoc. M., Buenos Aires, Argentina. (Elected Oct. 12, 1925.) (Age 36.) Asst. to Gen. Mgr., Ferrocarril Terminal Central de Buenos Aires. Refers to J. L. Lytel, O. McDermith, R. A. Marshall, G. Schobinger, B. S. Thayer.

HUNTER, RALPH WALDO, Assoc. M., U. S. Veterans' Hospital Oteen, N. C. (Elected Dec. 14, 1925.) (Age 50.) Refers to C. E. Burleson, G. O. Consoer, W. D. Faucette, G. P. Morrill, J. E. Willoughby.

ROSS, CONRAD MILTON, Assoc. M., Charleston, W. Va. (Elected Jan. 18, 1926.) (Age 36.) Asst. Engr., Public Service Comm. of W. Va. Refers to E. N. Blackwood, T. F. Boltz, H. L. Campbell, R. P. Davis, J. T. Moore, T. A. Polansky, W. D. Sell, R. W. Smarr.

FROM THE GRADE OF JUNIOR

AMBERG, GUSTAV ADOLF, Jun., Memphis, Tenn. (Elected Nov. 26, 1923.) (Age 32.) Pres., G. A. Amberg Co., Inc., Contrs. and Engrs. Refers to R. M. Beanfield, C. D. Marx, R. Matthew, B. B. Somervell, F. H. Tibbetts, R. G. Wadsworth, C. B. Wing.

AULL, LUTHER BACHMAN, Jr., Jun., Union, S. C. (Elected Jan. 19, 1925.) (Age 32.) Res. Engr., State Highway Dept., Columbia, S. C. Refers to R. B. Cureton, T. F. Hickerson, T. K. Legare, L. W. Pollard, T. Saville, R. M. Trimble, J. G. Wardlaw, Jr.

FELDMANN, EDWARD HENRY, Jun., White Plains, N. Y. (Elected April 18, 1927.) (Age 30.) Asst. Engr. with W. W. Young, Cons. Engr. for Westchester County San. Sewer Comm. Refers to A. M. Brosius, G. W. Gardner, C. A. Latimer, F. J. Laverty, C. J. Suhr, A. M. Wyman, F. C. Zeigler.

GEIGER, CHARLES DAVID, Jun., Paterson, N. J. (Elected Nov. 10, 1930.) (Age 27.) With William Ferguson. Refers to G. Ferguson, J. W. Ferguson, S. E. Greydanus, H. J. Harder, G. Hewitt, W. M. Jones, E. M. Lee, J. E. Torrey, P. L. Voss, F. J. Wright.

JUNG, WERNER FRED, Jun., Cleveland, Ohio. (Elected Dec. 3, 1928.) (Age 32.) Asst. Engr., Ohio State Highway Dept. Refers to E. C. Blosser, R. C. Chaney, E. T. Frost, C. T. Morris, J. R. Shank.

LERCHEN, FRANK HUGH, Jr., Jun., Madden Dam, Canal Zone. (Elected July 27, 1931.) (Age 27.) Concrete Placing Foreman, Madden Dam. Refers to A. J. Ackerman, R. C. Booth, C. F. Calhoun, A. Donaldson, C. P. Dunn.

ROUSCULP, JOHN ALLEN, Jun., Columbus, Ohio. (Age 32.) Senior Designing Engr., Div. of Eng., City of Columbus. Refers to R. A. Allton, O. Bonney, E. G. Bradbury, J. H. Gregory, J. C. Prior, C. E. Sherman, R. H. Simpson.

The Board of Direction will consider the applications in this list not less than thirty days after the date of issue.